The Observational Method – Lessons Learnt and Future Potential

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ABSTRACT: Ralph Peck (1969) in his seminal Rankine lecture noted that 'anyone who practices the application of soil mechanics needs to be fully aware of the potential value of the Observational Method (OM) and needs to understand the prerequisites for its success'. Peck noted how Karl Terzaghi reflected on his methods for solving practical problems during the early development of the OM. The OM is a powerful practical technique which can provide significant advantages for many civil engineering projects. When used effectively it can: create major time and cost savings, increase safety and reduce project risks, enhance collaboration between client, design and construction teams, and enable the construction industry to learn from experience and improve future practice. Unfortunately, despite this compelling list of advantages, the OM continues to be significantly under-used. Two key constraints are commonly used contract conditions and industry culture. This lecture will explore the advantages and the limitations of the OM, using several case histories to illustrate the lessons learnt and review how key constraints (both technical and commercial) can be overcome. Recent OM applications highlight the immense power of new technologies in instrumentation/monitoring and numerical analyses to facilitate near real-time back-analyses. These technologies provide a platform for exciting new developments whereby OM could provide: (1)'Intelligent digital twins' for our future built infrastructure (2) The basis for 'Adaptive Designs' which could enable a more effective balance between resilient infrastructure (to withstand climate change) and minimising cost and carbon during construction (3) Accessible data bases on full-scale performance, providing an evidence base for more informed selection of design parameters for future projects. The paper concludes with recommendations for OM practitioners.

KEYWORDS: Observational Method, Instrumentation & Monitoring, Case Histories, Risk Management, Safety, Real-Time Back Analysis, Serviceability Limit State.

1 INTRODUCTION

1.1 *Scope of Paper*

Terzaghi's unique and far-reaching contributions to the development of applied soil mechanics are legendary and profound, and this included the pioneering development of procedures which are now known as the Observational Method, OM, which is the subject of this invited paper. This paper will begin with a brief summary of Terzaghi's and Peck's seminal contributions and then discuss the following:

1. Benefits of OM and the range of applications
2. Achieving agreement to use the OM
3. Key requirements and limitations
4. Practical applications through 3 case histories
5. Lessons learned
6. Future potential

1.2 *Terzaghi and Peck's pioneering developments*

Peck (1969) in the introduction to his Rankine lecture on the Observational Method, OM, stated that '*I believe we owe its (i.e. the OM) formulation and development as a systematic procedure to Karl Terzaghi.'* Terzaghi (1961), in his lecture on the past and future of applied soil mechanics, warns about the misuse of soil mechanics, stating that there are – '*many situations when the geological conditions preclude the possibility of making an accurate performance forecast'*. In the 21st century with the widespread use, and mis-use, of finite element analysis, the wisdom of Terzaghi's words remains relevant for geotechnics practitioners. Terzaghi (1961) outlined four case histories where:

The geological conditions preclude the possibility of securing in advance of construction all the essential information required for adequate design. If this condition prevails, sound engineering calls for design on the basis of the most unfavourable assumptions compatible with the results of the subsoil explorations. This rather uneconomical procedure can be avoided only on the condition that the project permits modifications in the design during or after construction in accordance with the results of significant observational data which are secured after construction is started. This can be called an "observational procedure".

One of the four case histories where the OM was used by Terzaghi was the 45m high Vermillion Dam in southern California. Figure 1 shows a plan of the Dam and several key features and observations. The dam rests on 65m thick fluvio-glacial deposits. The cost of a cut-off to bedrock was considered prohibitive and a reliable prediction of seepage, and the associated risk of piping failure, through the fluvio-glacial deposits (given the erratic stratification in this depositional environment) was impractical. The design was developed on the basis of relatively optimistic, most probable' conditions, together with a network of observation wells to observe actual seepage patterns. Localised water seepages at ground surface were observed shortly after reservoir filling (shaded areas in Figure 1). The danger of piping failure in these areas was solved 'at very moderate expense' by a surface covering of inverted graded filter. This simple contingency measure was rapidly implemented from stockpiles of filter materials placed in downstream areas prior to reservoir filling. Even with the advances which have taken place in geotechnics since the Vermillion Dam was constructed, Terzaghi's application of the OM would still represent an optimal solution given the intrinsic uncertainties posed by the site geology.

Following Terzaghi's pioneering use of the OM during the middle decades of the 20th century, Peck (1969) then gave the first formal description of the OM during his seminal Rankine lecture. Peck outlined eight steps, Table 1, which he considered were required for a complete application of the OM. He also identified two main categories, termed 'ab initio' (where OM is envisioned from the start of a project) and 'best way out' (where some unexpected development occurs creating a project crisis).

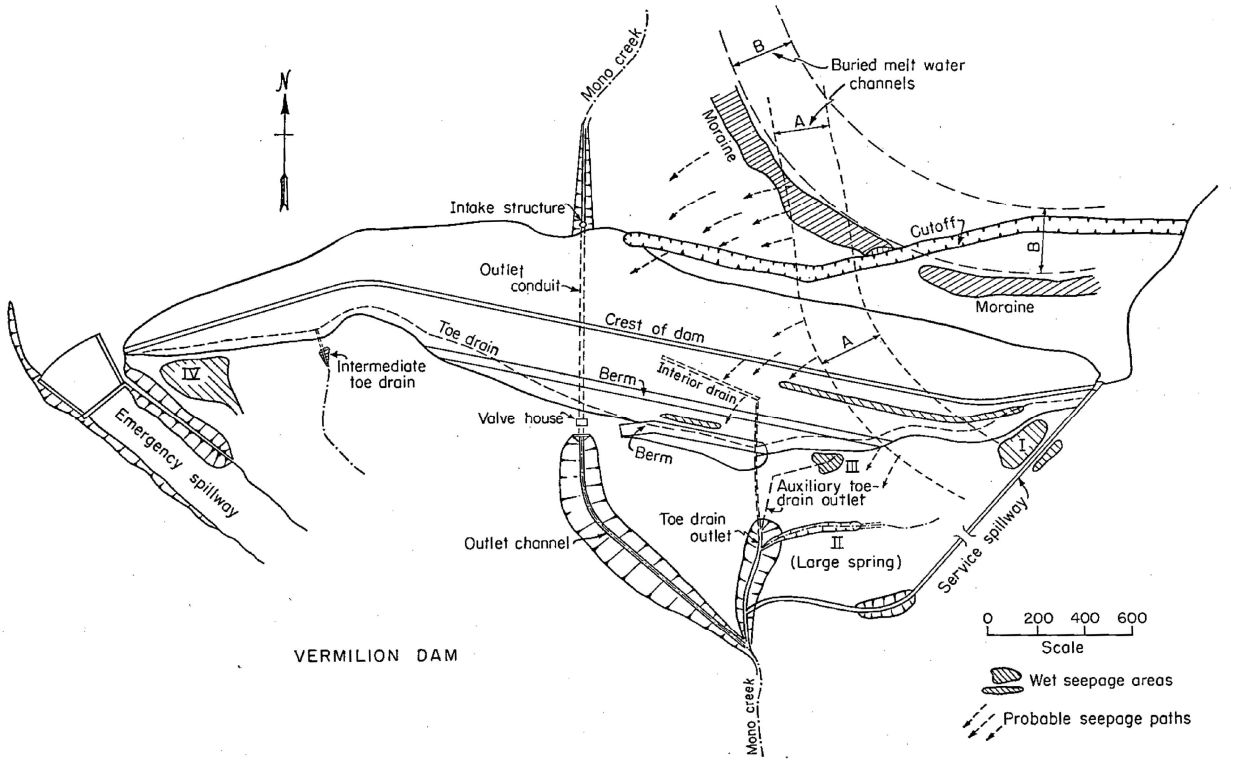


Figure 1.Plan of Vermillion Dam. In Areas I to IV water came out of the ground while the reservoir was being filled for the first time (after Terzaghi, 1961).

Peck's eight steps remain a topic of discussion amongst OM practitioners, and reflects the dichotomy faced by any attempt to rigidly define requirements for an inherently flexible approach; particularly given that the OM addresses uncertainty and demands the exercise of experienced engineering judgement. The authors have found that Peck's eight steps remains an excellent guide for OM applications. Peck's approach was pragmatic and recognised that each potential OM application needed to be considered on an individual basis.

Table 1. Peck's Eight steps

(i)	Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
(ii)	Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role.
(iii)	Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions
(iv)	Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
(v)	Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.
(vi)	Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis
(vii)	Measurement of quantities to be observed and evaluation of actual conditions.
(viii)	Modification of design to suit actual conditions.

NB – the degree to which all these steps can be followed depends on the nature and complexity of the work.

1.3 *OM benefits and range of applications*

In the decades following Peck's Rankine lecture, numerous case histories have shown that the Observational Method is a powerful, practical technique which, when used effectively, can deliver:

- Major cost, time and carbon savings;
- Enhanced safety, for example, by focusing on good communication and clearer procedures and control during construction;
- More effective collaboration between designers and constructors as well as the client;
- Opportunities for driving innovation and more effective learning from projects.

Table 2 provides a brief summary of cost and time savings for a small selection of major projects. Despite these compelling and demonstrable benefits, the OM remains significantly under-used. The reasons for this are discussed in section 2 below.

The technical basis for the OM is well established, and a wide range of guidance is available. CIRIA report 185, Nicholson *et al* (1999) provides detailed discussion and definition of the OM and relevant processes for controlling implementation. This CIRIA guide is currently being updated (CIRIA P3295). The current version of Eurocode 7 (EN-1997) explicitly includes provision for using the OM (specifically through clause 2.7). Szavits-Nossan (2006) compared different applications, the nature of uncertainty and how it has been addressed. More recently Lacasse and DiBiagio (2019) discussed the application of modern analysis techniques (and specifically Bayesian updating) in the context of implementation of the OM. Powderham and O'Brien (2020) describe the practical application of the OM through a dozen

case histories which cover a range of different geologies, design constraints and construction challenges.

The OM has been used successfully across a wide range of applications; Table 3 provides a summary of different design situations and case history examples. OM applications for tunnelling and propped/anchored retaining walls are often discussed in the OM technical literature. However, it is important to recognise that the OM has been applied to a much wider range of design scenarios as indicated in Table 3. It can be argued that some design applications such as dewatering/groundwater depressurisation cannot be adequately implemented without the use of the OM.

Table 2. The OM - examples of benefits

Project	Project Type	Location	Benefits
Northern Line Extension.	Retaining wall, deep excavation	London, UK	3months; US $5 M.
Boston Central Artery, Contract 9A.	Jacked Tunnel below operating railway	Boston, USA	12 months; US$ 300 M.
Liantang	Tie-back retaining wall + tunnel portal	Hong Kong, China.	3.5months; US$ 43M.
DTSS2	Retaining walls / deep shafts	Singapore	8months; US$3.4M.
Dublin Port Tunnel	Soil nailed steep cut slope	Dublin, Eire	3months; US $20 million.
A406 highway	Retaining wall	London, UK	6 months; US$ 1.3m.
Batheaston Bypass	Retaining wall	Bath, UK	9 months; US$1.2 M.

NB- typical time savings of between 10% and 20% from original project critical path. Cost savings typically exceed initial 'investment' costs of using OM (e.g. extra design inputs, mobilisation of contingency measures to site, instrumentation / monitoring, additional site staff, etc) by more than a factor of 20.

2 ACHIEVING AGREEMENT TO USE THE OM

2.1 *Key issues*

In the author's experience, by far the most difficult aspect of implementing the OM is the first step, which is achieving agreement to use the OM (O'Brien et al, 2026a). Powderham and O'Brien (2020) describe the practical application of the OM through a dozen case histories which cover a range of different geologies, design constraints and construction challenges. Importantly, each case history includes a description of how agreement was achieved to use the OM.

Despite the compelling list of advantages noted above, the OM continues to be significantly under-used. Typically, the main reasons for OM not being used are:

- Contracts (particularly between Main Contractor and Client) which inhibit the use of OM
- Industry culture, which leads to poor management of risk and opportunity, together with the key organisations (such as: designer, checker, contractor, etc) wishing to remain within their respective silos and comfort zones
- Design Checking / Assurance - independent design checkers are reluctant to approve OM, since it requires the ability to modify the design during construction on the basis of the observed behaviour.

Table 3. Examples of different OM applications

Design/Construction Scenario	Case History	Comments
Retaining Walls	Pye *et al*., 2022	Elimination of temporary props
Deep Shafts	Liew *et al*., 2016	Simplification of construction sequences and reduction of temporary works
Tunnel Jacking	Powderham and O'Brien, 2020	Control of jet grouting, ground freezing + tunnelling
Pile Groups	Powderham and O'Brien, 2020	Control of pile group rotation during major structural lifting
Embankments	Wu, 2011	Control of rate of embankment construction
Ground Improvement	Solera and Lehane, 1992	Control of grouting to limit groundwater flows
Protection of Existing Infrastructure	Powderham and Tamaro, 1995	Control of Tunnelling (timing + sequence) to limit differential building settlement
Dewatering	Roberts and Preene, 1996	Optimising cost-effectiveness of dewatering
Offshore Structures	Lacasse and DiBiaggio, 2019	Extend operational life of offshore structure
Dams	Jamiolkowski, 2014	Stability of existing Tailings Dam
Protection of Ancient Monuments	Burland et al, 2013	Control amount + location of soil extraction to stabilise tower
Compensation grouting to reduce tunnel induced settlement	Harris *et al*., 1996	Control amount + location of grouting to minimise differential settlement

2.2 *Contract Conditions*

Commonly used contract conditions are often the main reason why the OM is not used. Table 4 provides a brief overview of different contract types and their compatibility with OM implementation.TC206, has published a contracts guide, aimed at facilitating wider application of the OM (at www.issmge.org/committees/technical-committees).

Fundamentally, the OM requires real collaboration and trust amongst the key organisations involved in its implementation. Unfortunately, adversarial behaviour, a silo mentality and a lack of trust are common across the construction industry. A shift to more collaborative behaviour requires, as a minimum, for contracts to stimulate:

i. closer co-ordination between client, contractor and designer to allow opportunities to be identified and developed;
ii. non-adversarial environment which encourages the required teamwork;
iii. a commercial framework that encourages the above, so benefits, in particular cost savings (note: these may be direct or indirect due to time savings) are shared equitably between parties.

The above requirements are not generally facilitated by the standard types of priced (lump sum or bill of quantities based) contracts which are often used in the construction industry. Hence, it is unsurprising that the OM is under-used. Experience indicates that the most effective stimulus to implementation of the OM is to include a Value Engineering (VE) clause in the contract, discussed below.

Table 4. Overview of Contract Types

Contract Type	Key Features	Collaboration between designer and Contractor	Opportunity for OM
Traditional (Design/Bid/Build)	Client appoints designer, design completed, successful contractor builds design	Very Limited. Typically, designer separated from Contractor	Very limited, unless a Value Engineering (VE) clause is introduced.
Design and Build	Client's designer prepares a 'reference design'. D&B team prepare tender design and if successful completes final design and builds project.	Potentially good, but intense time pressure during tender may limit opportunity to build rapport and trust.	More opportunity than traditional, but Client approvals and independent checkers may hinder. May not be in commercial interests of Contractor or designer to pursue OM, unless incentivised through a VE clause.
Early Contractor Involvement (ECI)	Reference design developed by appointed Contractors during 'Stage 1' of ECI process. Stage One ends with the agreed scope, prices, programme and risk allocation for the remaining design and construction in 'Stage Two'. Stage Two is often a target contract.	Typically, better than conventional D&B, due to reduced time pressures and opportunity to seek and develop innovation	Very good potential, since additional time during Stage 1 of ECI provides greater opportunity to build trust between parties.
Alliancing	Multi-party delivery framework which focuses on a co-operative process to promote trust, risk and responsibility sharing. Alignment of commercial interests drives collaboration.	Very good. Innovation promoted through more collaborative environment. Positive interaction between all parties encouraged.	Potentially excellent. The alignment of commercial interests and risk sharing is conducive to OM. Potential development of long-term relationships, should further enhance OM opportunities.

2.3 *Value Engineering*

The implementation of the OM requires substantial extra effort during the early project stages, especially by the contractor and designer (for example, evaluation of alternative design measures across a range of conditions from most probable to worst credible and associated extra analysis, design and planning for contingency measures, etc). Hence, there must be some commercial incentivisation to encourage the parties to begin considering OM implementation, otherwise the OM will be a non-starter irrespective of perceived technical benefits, (Tran and Molenaar, 2012). Value engineering is the term given to a contractual mechanism that encourages the contractor to suggest changes to the original conventional 'base case' design. In the authors' experience the inclusion of a value engineering clause has been the crucial catalyst for the OM to be implemented. The key features of this type of clause are:

(a) each party is encouraged to propose changes that provide a benefit to the project, such as a reduction of time, costs, improved safety, etc;
(b) the client can reject a proposal, but the reasons must be given and a resubmission allowed, provided it addresses the reasons for a previous rejection;
(c) the parties receive a fair share of the savings, based upon the level of risk taken by each party (the percentage split should be specified in the value engineering clause).

The main parties will be the contractor and client. However, the contributions by the designer, checkers and specialist sub-contractors should also be appropriately recognised.

2.4 *Gaining Stakeholder Support*

Stakeholder support is a critical ingredient for successful implementation of the OM. Moving key organisations out of their silos and into more positive collaborative working relationships is a crucial step in the use of the OM. The OM demands commitment by all stakeholders and is fundamental to ensuring success in any application. Consequently, when there is no overriding driver for change, for example an 'ab initio' application, there is little incentive to depart from convention or 'business as usual'. The parties are inclined to remain within their respective silos or comfort zones. Ab initio applications are typically developed after a preliminary design is developed and project constraints raise specific challenges, (e.g., Powderham and O'Brien, 2020). By contrast, with projects that have reached a crisis, there will be a marked absence of comfort generally. This will be evident in polarised and often strongly conflicting perspectives between the parties. Here the OM would be proposed as the 'best way out' solution. Peck (1969) noted that such OM applications were more common, indicating that resistance to change in the absence of a crisis is quite common. Even with a crisis as a catalyst, it often takes sustained and convincing advocacy by an OM champion to secure the 'buy-in' by the other parties. They will probably be in varying degrees of disagreement and will therefore not readily reach a consensus that the OM offers the best way forward. It will test the commitment, confidence, resilience and ability of the OM champion as it can take detailed development, strong advocacy and substantial time and effort to reach agreement. The key factors involved are summarised in Table 5. There is a wide variation in degree of challenge involved in gaining agreement. The simplest situation is usually where only the temporary works are affected, and there is no potential impact on existing infrastructure. The most complex situations are where the OM potentially influences the permanent works and/or existing infrastructure and typically leads to a larger number of stakeholders who need to agree to OM being used.

2.5 *Design Checking / Assurance*

Checking procedures are becoming more complex in many parts of the world, especially for major projects. Hence, there is an increasing risk that independent checking and approval procedures becomes a factor which prevents OM being implemented. Clients need to be aware of this pitfall and ensure when appointing checkers that innovative alternative ideas, including OM, are fairly and objectively assessed by suitably competent and experienced professionals. The independent checking scope and timing are important when implementing the OM.

Table 5. Key Factors in Achieving Agreement to use the OM

Factor	Issues	Comments
Convincing Business Case	Practical benefits need to be clear and communicated to stakeholders	May include costs, time and carbon savings, safety improvement, technical risk reduction. Advantages need to be compared against conventional base case.
Sound Technical Basis	Often OM involves some form of ground-structure interaction, either new build or existing infrastructure, or both.	Both geotechnical and structural aspects need to be understood. Hence, multi-disciplinary input + competence required. Must be evident to stakeholders.
Risk Management	Maintaining + demonstrating an acceptable level of safety is essential. A wide range of risks need consideration, both technical + commercial.	Perceptions of risks can vary across different stakeholders. OM practitioners need awareness of these and appropriate mitigations.
Trust	Critical for any OM application.	Trust has to be earned + requires time + good interpersonal relationships.

3 KEY REQUIREMENTS AND LIMITATIONS

3.1 *The essence of the OM*

The essence of the OM can be simply expressed as: '*the key to success in applying the OM is in identifying the critical observations and having the means to obtain and act upon them in a safe and timely manner*', Powderham and O'Brien (2020).

The above simple summary leads to four key requirements for the OM, which are:

1. The critical observations can be obtained reliably and within acceptable timescales
2. Timely contingency plans can be implemented.
3. The ground and structure(s) (and any associated interactions) are not prone to sudden or progressive collapse
4. There is stakeholder support

The rigour demanded by these requirements is onerous. These requirements must be met for all foreseeable conditions, i.e. varying from 'worst credible' through to 'most probable', and there is a planned course of action for every foreseeable eventuality. Superficially these factors may seem to be independent, however they are essentially a combination of the first two. In the OM a fundamental criterion is the associated time factor. It must be possible to detect adverse trends in advance of an unacceptable event and be able to implement a planned contingency to safely deal with it.

The third factor also relates to the time factor, and requires a fundamental understanding of credible collapse mechanisms, potential load paths and likely associated timescales. Typically, deep and broad expertise in geotechnics and structural engineering is required, together with appropriate construction experience.

Finally, stakeholder support is vital because OM applications demand clear communications and close teamwork throughout the OM implementation. Lack of support or commitment from any member of the project team could undermine this essential communication and critically impair risk management. Any lack of trust or ownership of specific tasks would unacceptably compromise the ability to obtain reliable measurements or implement contingency measures in a timely manner. This trust and commitment are particularly important between designer and contractor.

3.2 *Key Limitations*

It follows that any inability or shortfall in the above four requirements means that the OM cannot be safely implemented on a project. A cardinal limitation of the OM is if it is not possible to change the design (in a timely manner) during construction, then this renders the OM a non-starter.

Table 6 summarises a few examples of where the OM could not be safely implemented. The critical constraints, either being structural details or groundwater conditions, meant failure / excessive damage could occur before an effective contingency measure could be implemented. At Limehouse Basin a preliminary trial highlighted the key risk, and for the T5 tunnels the risk was anticipated from the known structural details for the tunnel. In the case of the multi-storey car park, MSCP1A, its sensitivity to small lateral ground movements was not anticipated. In fact, two apparently similar reinforced concrete structures, MCSP1 and 3, did not suffer any significant damage (even though they were subject to larger ground movements). The high sensitivity of MSCP1A is a reminder that differences in structural detailing can significantly change a structure's sensitivity to ground movements. Nevertheless, as noted by Powderham et al (2004), it remains a far outlier in terms of building sensitivity to ground movement.

Table 6. Limitations of the OM, example projects where OM could not be safely implemented

Project	Structure	Key Issue	OM Challenge	Comments
Heathrow T5 tunnels	Tunnel with pre-cast expanded lining, unbolted	Decompression + sudden collapse of lining	Obtain critical observation (total hoop stress)	Local works to eliminate decompression risk, OM used for bulk of basement works.
Limehouse Basin temporary cofferdam	Multi-propped sheet pile wall	Bending failure of sheet piles	Insufficient time to implement contingency	A mid-height soft prop trial indicated wall failure may occur within 7 days
HEX CTA bored tunnels	Multi-storey car parks (MSCP) 1, 1A, 3	Shear + Flexural cracking of reinforced concrete columns.	Excessive cracks induced rapidly + at unusually small movements (5-10mm) at MSCP 1A only	MSCP 1+3 undamaged, MSCP1A had unique sensitivity to small horizontal ground movements.

Note: MSCP1A sensitivity due to combination of stiff columns built monolithically into first floor slab and isolated foundations (lacking horizontal restraint in form of ground beams)

3.3 *Instrumentation, Monitoring and Observations*

Reliable instrumentation, timely interpretation of monitoring data and associated site observations are essential in the application of the OM. As discussed by Peck (1972), the term observation includes instrumentation, but instrumentation by itself is inadequate. To quote Peck: *"even with the most sophisticated instrumentation, other types of observation are essential"*. These other observations are necessary to be able to interpret the instrumentation data and understand the link

between cause and effect. These other observations may include: condition and historical surveys of existing infrastructure and buildings; careful logging of excavations by experienced engineering geologists; detailed records of construction activity; climate variations, etc. The establishment of comprehensive records of construction activity is an essential, but often over-looked, element of the overall monitoring system. Without these records interpretation of ground or ground-structure interaction will be impossible. The identification, timely collection, interpretation and communication of the critical observations are vital. Every instrument deployed should answer a specific question. The effective implementation of the OM requires the identification of the primary instrumentation which will be used for measurements of the key parameter to be used for controlling the works and comparing these measurements against the agreed performance limits. It must be possible to identify trends and separate key construction induced events from background and secondary effects. This means that some of the instruments should be installed well before construction commences, so environmental background effects (eg daily and seasonal changes) can be understood.

Some redundancy should be included to ensure the continuity of readings in case of an instrument malfunction. A secondary system should be established as a back-up to the primary and avoid over-reliance on one type of system. However, it is important to avoid installation of excessive amounts of instrumentation since this risks data over-load and a loss of focus on the critical observations. Table 7 provides a simple checklist of key issues which should be considered when developing a monitoring plan for the OM.

4 CASE HISTORIES

Three case histories have been selected. These cover a range of project scales from a small regional project (case history 1) through to one of Europe's largest projects, HS2 (case history 3). A range of application types are also discussed: an embankment on soft clay (case history 1), dewatering for a deep excavation (case history 2) and a propped retaining wall (case history 3). These case histories do have some common features despite the huge differences in project budgets:

- All had significant uncertainty, associated with the site geology within the context of the planned works;
- It was unlikely that additional ground investigations or analyses would have eliminated this uncertainty;
- All the case histories involved the application of OM through progressive modification (Powderham, 1998) and highlight the flexibility of the OM both to effectively manage risks and deliver considerable benefits (in terms of cost, programme, carbon, etc).

4.1 *Case History 1, Dearne Town Embankment – soft clay consolidation*

4.1.1 *Introduction*

Embankment construction on soft clay is a classical problem in applied soil mechanics. It has benefited from much research effort (e.g. Leroueil and Hight, 2003) and recent developments in advanced analysis. Nevertheless, there are significant uncertainties associated with predicting the time required for soft clay consolidation and associated increase in undrained shear strength. Perhaps the greatest uncertainty which affects many natural soft clays is the nature of the soil fabric (e.g. the continuity of thin partings of silt or sand) and its impact on the mass permeability of the clay during consolidation. This case history gives an example of the application of the OM for a small regional highway project, near Barnsley, Yorkshire, U.K. The project's overall construction programme was dictated by the assumed time for soft clay consolidation beneath four metre high bridge approach embankments. The contract specified a 44-week time period to allow consolidation of the soft alluvial clays. This delay period had serious financial implications for the contractor, given the assumptions which had been made for the tender. The senior author acted as the contractor's technical adviser for this OM application which was a 'best way out' application.

4.1.2 *Ground and groundwater conditions*

The available ground investigation data was limited to routine sampling and basic laboratory testing. This indicated a surface layer of soft alluvial clay, up to 5m thick, overlying medium dense to dense glacial sands and gravels. Groundwater varied seasonally but typically was about half a metre below ground surface level. The original designer's assessment of the clay's coefficient of consolidation was solely based on laboratory oedometer testing. This data indicated a range of Cv values varying from about 0.7 to 7 m2/yr, and even an optimistic interpretation of this data indicated Cv values of between 4 and 5 m2/yr (which would mean the delay period for consolidation would be close to the specified value). However, published experience of consolidation of alluvial clays suggested that the oedometer data would be mis-leadingly low, (Rowe, 1972). There was limited time or budget to attempt to carry out a more appropriate and sophisticated site-specific sampling and testing regime. Nevertheless, static cone and piezocone testing (CPT and CPTU), carried out during a weekend just after the contractor gained access to the site, provided valuable additional data. This data indicated the following:

- Most importantly, the soft clay had a pronounced macro-fabric; as shown on Figure 2 numerous silt and sand partings were identified, although the continuity of these higher permeability layers was uncertain;
- The overall thickness of the alluvial deposits was similar to that indicated by the designer's boreholes. However, the upper half metre was a desiccated crust of firm to stiff clay, and the lower half metre was a silty/sandy deposit, hence the overall thickness of the 'problem soil' was less than indicated by the boreholes.
- Undrained shear strength was variable, with minimum strengths locally about 10 to 15 kPa (these weakest layers being quite thin, circa half metre), and 'average' undrained strengths between about 20 and 25 kPa.
- These CPT derived undrained strengths were typically higher than indicated by the designers laboratory tests (unconsolidated undrained triaxial tests on thick-walled driven tube samples), and also indicated the soft clay was lightly over-consolidated (which contrasted with the designer's assumption that the Holocene age clay was normally-consolidated)
- Conditions along the southern approach embankment were more favourable than for the northern approach embankment (which indicated early access for bridge construction could be made via the southern embankment)
- CPTU dissipation tests, indicated large variations in consolidation coefficients with a nine fold variation between minimum and maximum values (reflecting variations between clayey and silty horizons). When approximate empirical corrections were made to the data (following Jamiolkowski et al, 1985), these indicated minimum and maximum Cv values of 10 to 90 m2/yr, with most values between about 15 and 25m2/yr.

Table 7. OM implementation, instrumentation and monitoring requirements.

Requirements	Comments
Identify critical observations	The primary criteria for controlling the works have to be clearly identified. The reliability, accuracy and precision of the measurements must be compatible with the assigned traffic light zones and timescales. Trends need to be monitored and related to actual construction activity.
Establish the instrumentation system	What, when and where instruments will be installed? Should be simple, reliable and use proven technology. Need to be installed early enough to establish background readings. Critical readings to be checked with an independent system. Robust protection and adequate redundancy to allow for construction damage or instrument malfunction.
Identify additional observations to enable monitoring records to be interpreted	Essential for comprehensive records of construction activities to be produced on a regular basis and at sufficiently short intervals during critical stages. These should be visual and present information unambiguously (e.g., construction photos and dimensioned sketches – in plans, long sections and cross sections).
Define roles and responsibilities within project team.	Project 'OM team' should include representatives from the designer and contractor. Frequency of monitoring review meetings must be defined (e.g., if necessary, daily during critical phases). Specific roles and responsibilities should be clearly established. Actions should be defined if trends demonstrate opportunities for beneficial design changes or, if adverse trends develop, the implementation of corrective contingency plans.
Method statement	Should give outline of how daily/weekly reports should be prepared. The monitoring plan should be clear and unambiguous. Construction control and simple flow charts should be produced to ensure key measurements and observations are recorded on a regular, planned and consistent basis. The OM team should lead preparation of these documents and ensure clear ownership of process. Examples of construction control charts and flow charts are discussed later in this paper.

The CPT/CPTU data was encouraging and indicated that there was a good chance that these alluvial clays would consolidate more rapidly than the designer had assumed. This opinion was also consistent with the experience reported from relevant case histories (Lewis et al, 1975), summarised in Table 8, were back-analysed 'field' values of coefficient of consolidation were considerably larger than laboratory oedometer values. Nevertheless, this more optimistic interpretation relied upon several approximate empirical correlations (with a wide scatter across the empirical data bases). At this stage neither the designer nor the client would relax the original specification requirements.

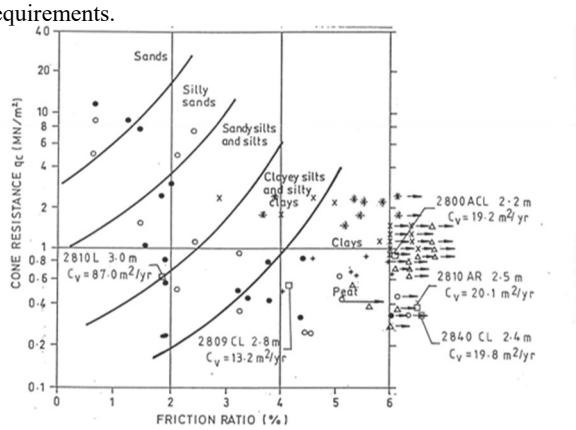


Figure 2.Dearne Town embankment, CPT classification chart

Table 8. Case history data, UK soft alluvial clays (TRRL LR 643)

Soil Layer	Plasticity Index (%)	Coefficient of Consolidation (m^2 / yr) Lab. Value	Field Value	$\frac{Cv\ field}{Cv\ lab}$
Brown Clay	55	0.5	11.3	22.6
Blue-grey Silty Clay	62	0.9	28.9	32
Dark-grey Silty Clay	35	1.4	65.3	46.6
Grey-brown sandy-silty clay	10 to 15	15.6	113.6	7.3

4.1.3 *Application of the OM*

Construction commenced as per the original design basis, but additional instrumentation was installed to enable the OM to be implemented. The primary instrumentation was very simple and comprised settlement rods along the embankment centre-line and survey pegs along the toe of the embankment. These were surveyed by the contractor's site surveyor once every two days throughout embankment construction. Secondary instrumentation comprised a dozen piezometers (4 piezometers installed at 3 sections). The decision to use piezometers as a secondary system only, was based on experience reported by Marsland and Powell (1977) which indicated that the ground movements (from the settlement rods and lateral movements from the toe pegs) were likely to be more representative of global behaviour – both in terms of the risk of the onset of instability and the overall rate of consolidation. Tavernas and Leroueil (1980) derived robust empirical correlations for the ratio of settlement to lateral movement both for undrained (prior to and after the pre-consolidation pressure was exceeded) and drained behaviour, summarised in Figure 3. Matsuo and Kawamura (1977) have related the ratio of lateral movement to embankment settlement to overall factor of safety, Figure 4. Figure 4 was used as a construction control chart, to ensure the embankments were constructed slowly enough to avoid undrained shear failure.

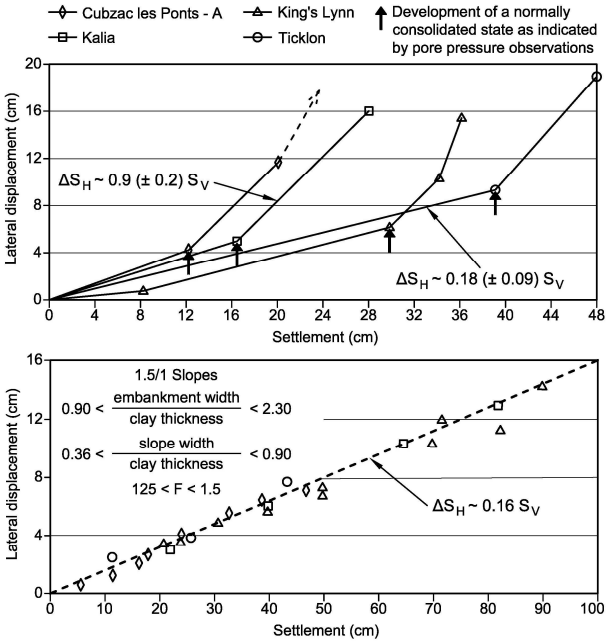
The contingency measure if the red trigger was breached was to place fill material at the embankment toe to form a berm (predesigned to provide an adequate factor of safety, assuming worst credible undrained strength based on the CPT data). Figure 3, together with the piezometer data, provided insight into whether the soft clay behaviour was predominantly drained or undrained.

The embankment was originally planned to be built in three stages of about 1.35m per stage, with a three-week consolidation period between each stage and each stage being built in two weeks; with overall embankment construction taking 12 weeks. However, the observational data during the first stage indicated small movements and excess pore water pressures; confirming the soft clay was lightly over-consolidated. Hence, the decision was taken to complete construction in two stages within an overall time period of 6 weeks. Figure 5 shows the recorded settlement data, for a representative section of the northern embankment, with consolidation being practically complete in less than ten weeks

(consistent with a Cv value of about 20m2/yr); the southern embankment consolidation was practically complete within a period of about 8 weeks. The client and designer were initially skeptical but following several presentations of the observational data they eventually agreed that the embankment consolidation was practically complete. Approval for the subsequent highway works, including bridge construction was given 30 weeks earlier than the original contract date.

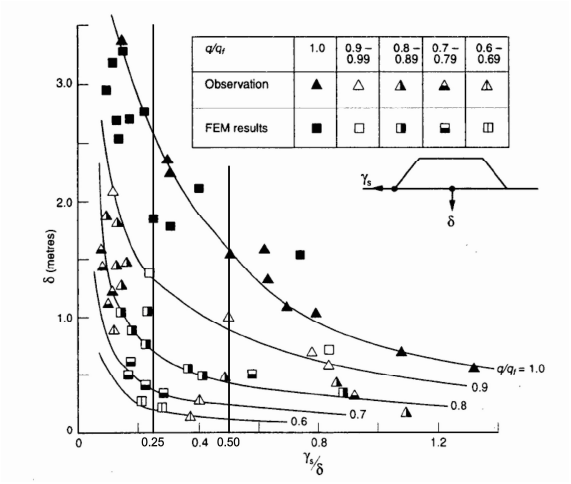
4.1.4 *Summary*

This simple OM application for a relatively small project provided significant benefits both for the contractor (who delivered a profitable project) and the client (since the project was completed early, for the benefit of the local community). The OM instrumentation, monitoring and reporting systems were very simple and could be managed by the contractor's site staff and their OM technical advisers. There was no sophisticated analysis; the analyses comprised simple limit equilibrium checks and one-dimensional settlement and consolidation analyses (with a combination of 'hand calculations' and a finite difference consolidation analysis).



Note: ΔS_H = horizontaL displcement; S_V = settlement

Figure 3.Case history observations of lateral displacement and settlement for embankments constructed on soft clay (after Tavernas & LeRoueil, 1980)



Key: δ = Settlement at the centre of the base of an embankment,
γ_s = Maximum lateral deflection at the embankment toe,
q = Embankment load, q_f = Embankment load at failure
Note: amber trigger, when lateral movement = 25% settlement; red trigger, when lateral movement = 50% settlement.

Figure 4.Dearne Town embankment, OM stability control chart, after Matsuo and Kawamura, 1977.

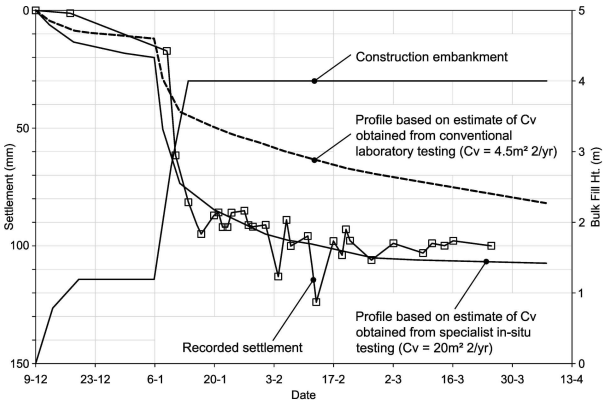


Figure 5.Dearne Town embankment. Construction, observed and calculated consolidation settlement vs time curves.

4.2 *Case History 2, Crossrail Blomfield Shaft – construction dewatering*

4.2.1 *Introduction*

Construction dewatering is an essential temporary works requirement for many civil engineering schemes. Terzaghi (1929 and 1955) recognised the substantial difficulties of predicting the behaviour of groundwater and wrote of the causes of major accidents and delays in foundation and underground construction: *"It is more than mere coincidence that most failures have been due to the unanticipated action of water, because the behaviour of water depends, more than on anything else, on minor geological details that are unknown".* This theme has been taken up by several eminent geotechnics practitioners, notably Rowe (1972) who describes a range of challenges arising from groundwater flow and changes in excess groundwater pressure. The problem has also been succinctly summarised as: "groundwater can seep through any theory"! Therefore, there is always a risk that any conventional design of a dewatering system may not be adequate. Hence, the use of the Observational Method (OM) should be seriously considered whenever construction dewatering is necessary, especially when the consequences of dewatering failure are unacceptable. This case history describes the application of the OM for one of the deepest excavations on Crossrail, the 42 m

deep Blomfield box which is located at Crossrail's Liverpool St station.

4.2.2 *Key aspects of design and construction*

The box is located in a congested urban area. To the north the Blomfield box is bounded by tunnels for the London Underground Ltd (LUL) metro and to the south by a large multi-storey building. The overall plan dimensions of the box are approximately 30 m long and 20 m wide. The box is initially constructed through the London Clay, but the final 5 to 6 m of the box is constructed through the Lambeth Group sequence (which includes silt and sand layers under high groundwater pressures). The retaining walls for the box are formed of 1.2 m diameter contiguous bored piles spaced at 1.4 m centres. The pile toe level is at 61 m ATD, i.e. about 52 m below ground surface level. Contiguous bored piles had to be used, rather than diaphragm wall panels due to the severe space restrictions at the site.

Given the excavation depth it was considered that attempting to form secant piles would be unsuccessful. The limiting depth for secant pile walls was about 25m, due to verticality tolerances and the piling rigs' ability to rotate a casing to cut into the installed primary piles. Adjacent to and penetrating into the box were a complex arrangement of sprayed concrete lined (SCL) tunnels, for the main tunnel, pedestrian access and escalator tunnels. Figure 6 shows a cross-section through the box, together with an outline of the geology. The main SCL tunnel shown is about 5 m from the box and its invert is located within the Lambeth Group. The SCL works for this tunnel also required dewatering through the Lambeth Group, however the SCL tunnel excavation would take place well before the excavation for the box (as part of a separate construction contract). The box was to be excavated top-down and the retaining walls were supported by six levels of permanent reinforced concrete beams and walers, Figure 6.

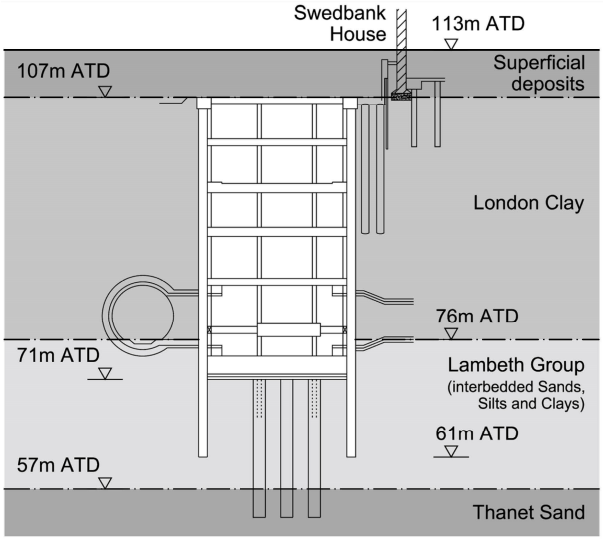


Figure 6.Crossrail Blomfield box and simplified geological profile.

4.2.3 *Ground and groundwater conditions*

The ground surface level is at about 113 m ATD. The ground conditions comprise 2.5 m of made ground and 4 m of Terrace Deposits, overlying a 30 m thick layer of London Clay. The Harwich Formation (about one metre thick), not shown in Figure 6, underlies the London Clay. The Lambeth Group (up to about 18 m thick) underlies the Harwich Formation. Both the Harwich Formation and Lambeth Group are highly variable, varying from very stiff clays to very dense sands and gravels. Groundwater level is approximately at 106 m ATD (the River Terrace deposits, predominantly sandy gravels at this site, are practically dry). At depth the groundwater pressure profile is non-hydrostatic, Figure 7, due to historical pumping from the Chalk aquifer, which underlies the Thanet Sand. The lower unit of the Lambeth Group (the Upnor Formation, which are typically silty sands), are in hydraulic continuity with the Thanet sands and Chalk aquifer.

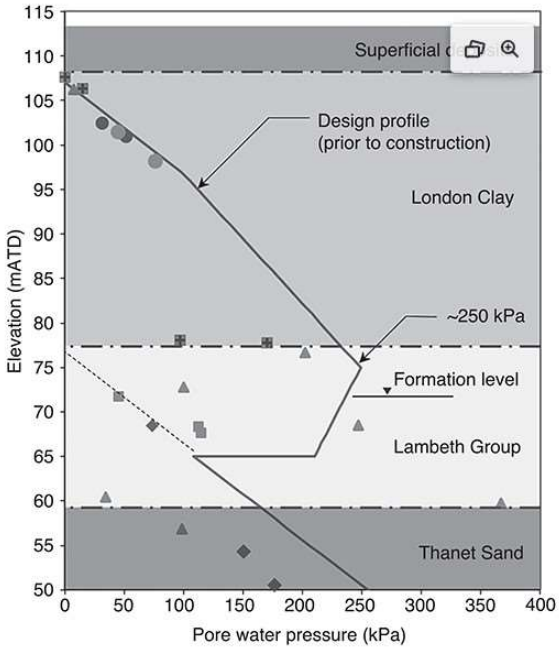


Figure 7.Crossrail Blomfield Box - groundwater pressures.

The groundwater pressure in the upper units of the Lambeth Group are relatively high, measuring up to 250kPa in the sand channels and the sand/silt horizons in the Laminated Beds. Hence, there was a risk of excavation base instability once the excavation reached an elevation of about 89 m ATD (i.e. as the overburden pressure reduced towards the groundwater pressure in the sand/silt layers, located a few metres below the Lambeth Group surface). The significant hazards associated with excavations in the Lambeth Group, and the complexity of the geometrical configuration of higher permeability layers of sands and silts interbedded with clay layers have been discussed in detail in CIRIA report C583. The Lambeth Group was formed in an estuarine environment; hence the nature of the ground and groundwater conditions can be highly variable, Figure 8.

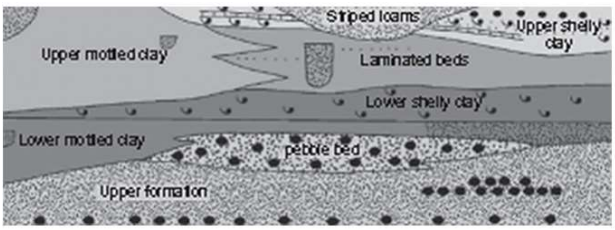


Figure 8.Potential variability of Lambeth Group (after CIRIA C583).

Figure 9 shows a photo of an exposure through the upper Lambeth Group, near the invert of one of the SCL tunnels, with interbedded clay, sand and silt together with a fault (at right hand side of photo). It was recognised that rapid changes in soil type could occur across horizontal and vertical distances of a few metres. Therefore, maintaining excavation stability though the Lambeth Group posed a considerable challenge. Base

instability risks would be increased during the final 5m of excavation due to the potential risk of ground unravelling or flowing through the gaps between the contiguous bored pile walls of the box (due to the high groundwater pressures in sand/silt layers). Hence, the critical challenge was how the groundwater pressures could be reduced in the sand and silt layers within the Lambeth Group.

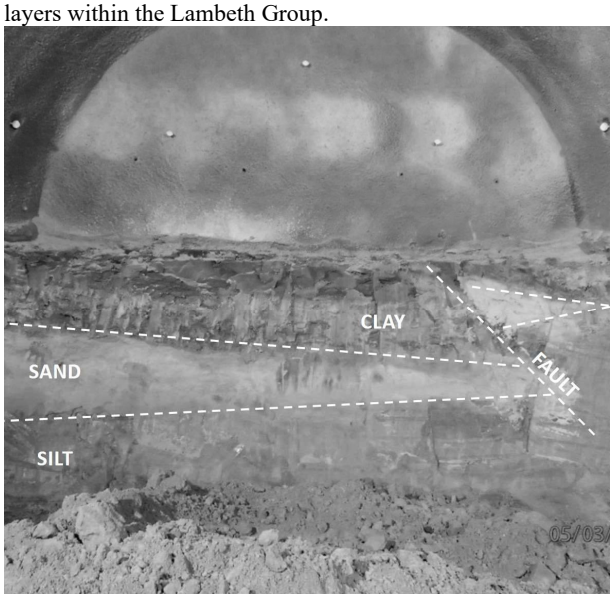


Figure 9.Exposure of Lambeth Group at invert of SCL tunnel. (Note: Fault at right-hand side and interbedded sand/clay/silt layers)

4.2.4 *Achieving agreement to use the OM*

The site, as noted above, was located within a heavily congested urban environment. This severely constrained ground investigations, due to the presence of existing buildings, underground utilities and structures. However, given the site geology, even if it had been possible to carry out extensive investigations, there would still have been considerable residual risks, as discussed above. Preliminary ground-structure interaction analysis also indicated that there would be considerable benefit in reducing groundwater pressures in the active zone behind the piled retaining walls, to make the prop forces more manageable. The use of closely spaced props towards the base of the excavation was anticipated to be counter -productive. Installing heavy props at depths of more than 30m raised safety concerns, given the limited space at the base of the box excavation. Also, installation of props and waling beams would be time-consuming and during this time period the Lambeth Group clays would be vulnerable to softening, owing to the short drainage path lengths with interbedded sand and silt layers. It was therefore decided that a dewatering system should be installed on both sides of the retaining wall, to lower groundwater pressures both inside and outside the box to acceptable values.

A conventional design of the dewatering system, prior to construction, could have been carried out and specified. However, this was considered to be unwise for several reasons. Firstly, the limitations associated with the available ground investigation data together with the potential variability of the Lambeth Group meant that there was a considerable difference between pessimistic (or worst credible) and optimistic (or most probable) ground/groundwater conditions. This meant that a conventional design could have been unsafe, if based on an optimistic interpretation of ground conditions, or difficult to build (and unnecessarily costly/time-consuming to implement) if based on a worst credible scenario. Secondly, dewatering design must consider several factors which are intrinsically difficult to quantify, for example, the lateral extent and connectivity of a layer to other permeable layers affects the effective zone of influence of a well and the time period a layer may keep producing water and how quickly groundwater pressure may be reduced and kept low with minimal pumping. Terzaghi's statement on the importance of "*minor geological details*" is particularly apt for the design of dewatering systems in the Lambeth Group. Worst credible assumptions included both high and intermediate permeability layers, together with some isolated pockets and some layers connected across a wide area (beyond the box footprint). The worst credible scenario required different types of wells and a relatively large number of wells to ensure the excavation would be safe throughout construction. Installation of all these wells in a single phase could have been counter productive, as they would obstruct the excavation and may have made subsequent works impractical. More optimistic sets of ground/groundwater conditions were considered as part of the evaluation of the benefits of implementing the OM. However, they could not be justified as a safe basis for a conventional design. At this site it would have been impractical to have attempted to identify the "most probable" conditions because of the potentially wide range of plausible conditions. The way forward was to implement the OM by progressive modification, based on a cautious estimate of ground conditions, and adjust appropriately on the basis of the observations.

Throughout the Crossrail project temporary works were the Contractor's responsibility. These included any dewatering. However, at the Blomfield box the dewatering would also affect the permanent works (e.g. deformation of the shaft walls and associated structural forces induced in the bored pile walls) and potential movement of adjacent infrastructure. Therefore, the OM was planned as an 'ab initio' application at the project inception to manage the implementation of the dewatering.

4.2.5 *Application of the OM*

A dewatering system can fail in several different ways, Roberts and Preene (1996). Groundwater flows may be excessively high or low, relative to the dewatering system capacity, which may then lead to failure of the dewatering system. Groundwater flow volumes, and their variation during excavation, depends upon several parameters, including: ground permeability; hydraulic gradient; connectivity between permeable horizons; their ability to recharge, etc. Permeability test data at the Blomfield site was limited, however, extensive permeability testing (both small scale lab tests, moderate scale borehole tests, and large-scale pumping tests) had been carried out in the Lambeth Group in adjacent sites. Permeability test data, Figure 10, indicated a permeability range of several orders of magnitude.

Based on previous experience and careful logging of site-specific boreholes, this variability of permeability was considered to be realistic.

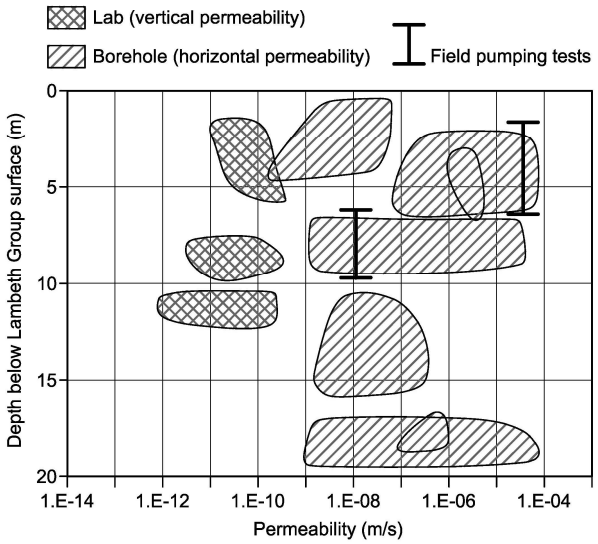


Figure 10. Permeability test values for Lambeth Group (lab, borehole and pumping tests).

To reduce the risk of failure, it was necessary to design a flexible set of dewatering systems which could cope with the range of permeability likely to be encountered, Figure 11. This included locally high permeability clean sands to lower permeability silts. The conceptual design included both deep wells and ejector wells. It should be noted that the fuzziness in the boundaries between different groundwater control techniques, shown in Figure 11, is deliberate and highlights the need for judgement when a boundary is being approached. An outline dewatering design, or base case was provided as part of the design drawings. This enabled the overall scope of the possible dewatering effort to be assessed for construction planning purposes and to ensure a suitable range of equipment would be mobilised to site. However, the contractor was permitted to modify the dewatering design provided the OM performance requirements were met. It was considered that the main uncertainty was the degree of connectivity between permeable horizons which depended on the fine detail of soil stratification. A fault zone was also anticipated across part of the box footprint, although its precise location was unknown. This greatly complicated the assessment of the hydrogeological regime, including the likely changes in groundwater pressures during excavation, and the effective radius of influence of wells. The arrangement of the construction contracts further complicated matters. Dewatering would also be required to construct the SCL tunnels (which were built under a separate contract to the Blomfield box), and it was known that this dewatering would be switched off before the box excavation commenced. Therefore, there was uncertainty associated with the rise in groundwater pressure which would take place once the tunnel dewatering had been switched off, and which would need to be subsequently managed under the box construction contract.

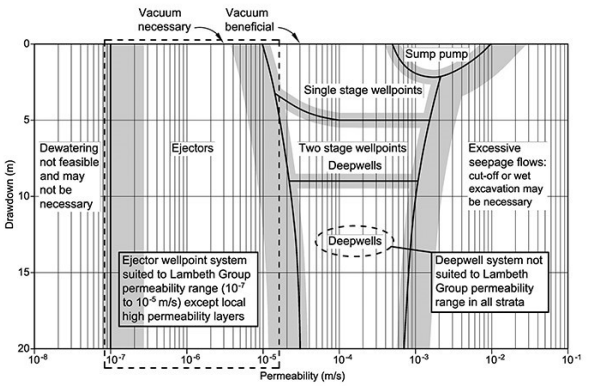


Figure 11. Permeability range for various pumped groundwater control techniques (modified after Preene, 2012).

For OM implementation, the primary monitoring system comprised 14 fast response vibrating wire piezometers (VWPs) and five standpipe piezometers to monitor the performance of the dewatering system. Secondary instrumentation for base stability comprised four magnetic extensometers. The critical observations were groundwater pressure; these had to be supplemented by the observations summarised in Table 9 to facilitate interpretation of the groundwater pressure data.

Table 9. Crossrail Blomfield box, observations to support monitoring of groundwater pressure.

Parameter	Method	Comment
Soil stratification	Logging during installation of all wells and instrument holes	Critical issue due to complex geology of Lambeth Group
Flow from individual wells	Flow metres	Important for selection of remedial options
Water quality and fines	pH/conductivity, check discharge tank	Water treatment necessary, removal of fines
Mechanical performance	Pump pressure, power supply	System maintenance, back-up needed

In addition, to the instrumentation it was vital for soil stratification to be carefully evaluated during the drilling of any boreholes for instrument or well installation, and as the excavation exposed the Lambeth Group deposits. Therefore, a senior geologist (as part of the OM team) was located on site throughout the OM implementation.

The installation of the dewatering wells and the instrumentation was planned to be carried out across several phases. This enabled the geology and hydrogeology to be better understood and also enabled the effectiveness of the dewatering system which had been installed to be checked on a progressive basis. This phased approach enabled the dewatering design to be modified on the basis of observed performance. It also enabled safety to be assured, whilst minimising the risk of slowing down excavation.

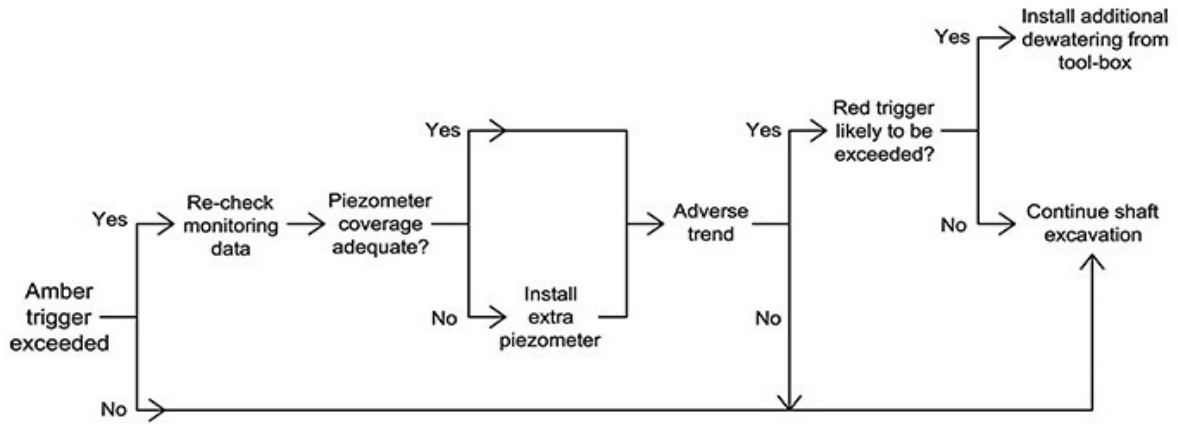


Figure 12. Crossrail Blomfield box – OM groundwater control chart

The installation of the VWPs was carried out in two different phases. Two VWPs were installed in the first phase, and before box excavation. These piezometers were located in the sandy layers of the Lambeth Group. The holes for piezometer installation were drilled using high quality rotary coring (triple tube wire line with mud flush) and the continuous core was carefully logged by an experienced geologist. This piezometer monitoring data provided a set of baseline readings, prior to box excavation influencing groundwater pressures. In the second phase of piezometer installation, 12 VWPs were installed and five standpipes. Some of the VWPs were installed behind the wall, by inclined drilling through the gaps between contiguous piles, when the excavation reached 83m ATD. Limited space meant that monitoring instruments and any wells for dewatering behind the shaft walls had to be installed by inclined drilling.

The groundwater control flow chart for implementation of the OM is shown in Figure 12. If an amber trigger was breached then more frequent monitoring was required, also the coverage of piezometers was re-assessed. If coverage was considered to be inadequate, then additional instruments were installed. If the red trigger was breached, then excavation would be stopped, and the dewatering system would be enhanced by installing additional wells from a toolbox of alternative systems. Selection of well type from the toolbox mainly depended upon the anticipated local ground conditions and their assessed permeability, Figure 11. The ground model (summarising the 3D variations in ground conditions) was progressively developed by the designer's site geologist on the basis of the site observations. The effectiveness of selected existing wells could also be enhanced (e.g. converting to vacuum enhanced wells). A key component of risk management was full time input by experienced design staff (who were part of the OM team), who were resident on site. This facilitated all monitoring data and site observations to be rapidly integrated and assessed in the context of the design requirements.

4.2.6 *Field Observations*

The installation of dewatering wells was carried out in two main phases: the first phase, with four deep wells installed; a second phase, with nine inclined and two vertical ejector wells. The purpose of the deep wells (installed when the excavation was only about 6m deep) was to facilitate a better understanding of the permeability and connectivity of the sand/silt layers in the upper units of the Lambeth Group. The ejector wells were installed when the excavation was at about 83m ATD, (i.e. when the excavation was 30m deep and about 7m above the surface of the Lambeth Group). During dewatering for SCL tunnel construction groundwater pressures were reduced to a minimum value of about 65kPa by November 2013. Following installation of the SCL tunnel lining, the tunnel dewatering was switched off and groundwater pressures increased during a six-month period (up to May 2014) to about 110 kPa. The internal deep wells, operating between July and September 2014 had a limited zone of influence, only reducing groundwater pressures outside the box by about 10 kPa (to about 100 kPa). The inclined ejector wells, operating from September 2014, were more effective, leading to a drop of 35 kPa in about 60 days (i.e. to pressures of about 65 kPa, below the amber trigger set for the final excavation stage of 70 kPa). The inclined ejector wells continued to operate effectively throughout the remainder of the works, and the amber trigger for groundwater pressures behind the bored pile walls was not breached. The gaps between the contiguous bored piles were infilled with concrete after each one metre excavation through the Lambeth Group. This provided temporary stability prior to construction of the permanent lining.

The main challenge during the later stages of the OM implementation, was the control of groundwater pressures inside the box, below the base of the excavation. When the excavation approached the Lambeth Group, at 76.5m ATD, the trends for the piezometer readings beneath the eastern side of the box indicated that the red trigger level was likely to be breached if no further actions were taken, Figure 13.

Figure 13 plots the variation of observed groundwater pressures, inside the box, against time, for the period up to November 2014. Groundwater pressures are shown for sand channels in the upper mottled beds (at 71.7m ATD) and in the underlying laminated beds (at 68.2m ATD) of the Lambeth Group. During 2013, when tunnel dewatering was operating, groundwater pressure variations exhibited a similar pattern to those beyond the box retaining walls. In the upper sand channel, tunnel dewatering reduced the groundwater pressures to slightly below 40 kPa. In the laminated beds, groundwater pressures were reduced to about 80 kPa. However, once tunnel dewatering was switched off there was a marked contrast between the sand channel and laminated beds, with groundwater pressures increasing within the laminated beds whereas they remained practically constant within the sand channel.

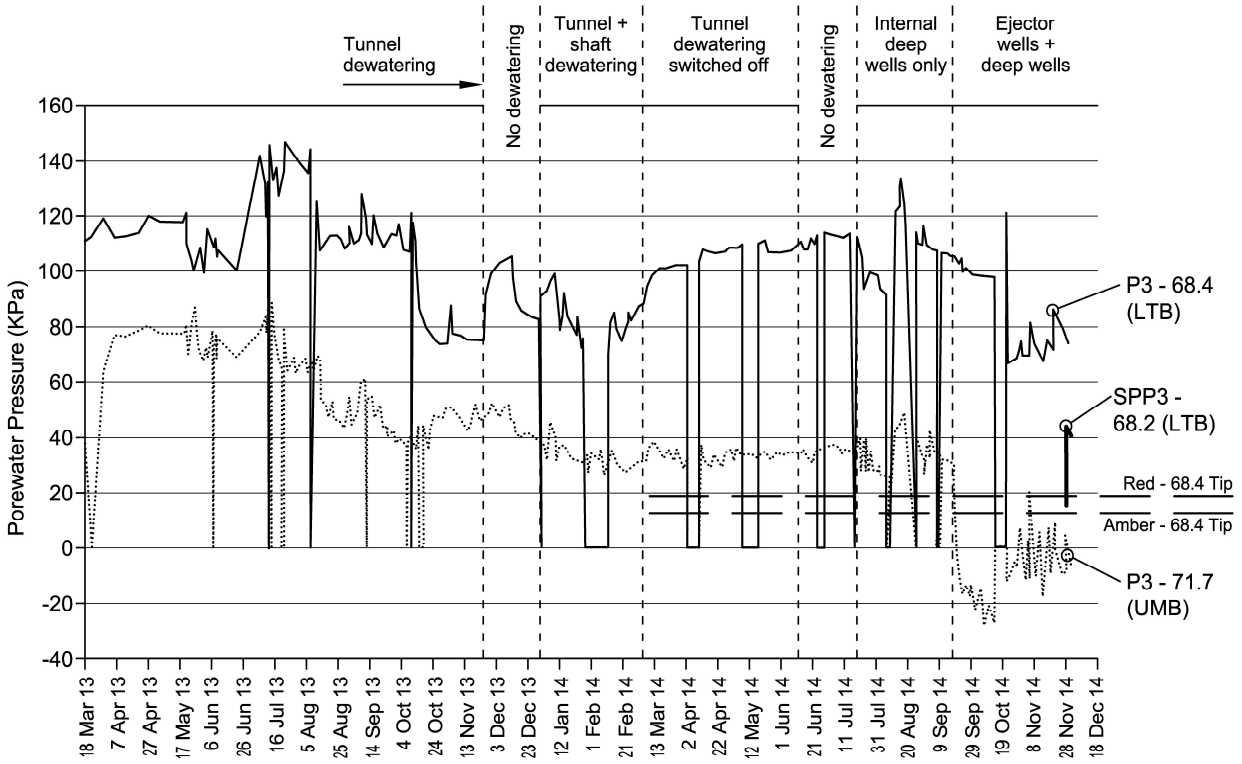


Figure 13. Crossrail Blomfield box, observed groundwater pressures, inside box, up to November 2014

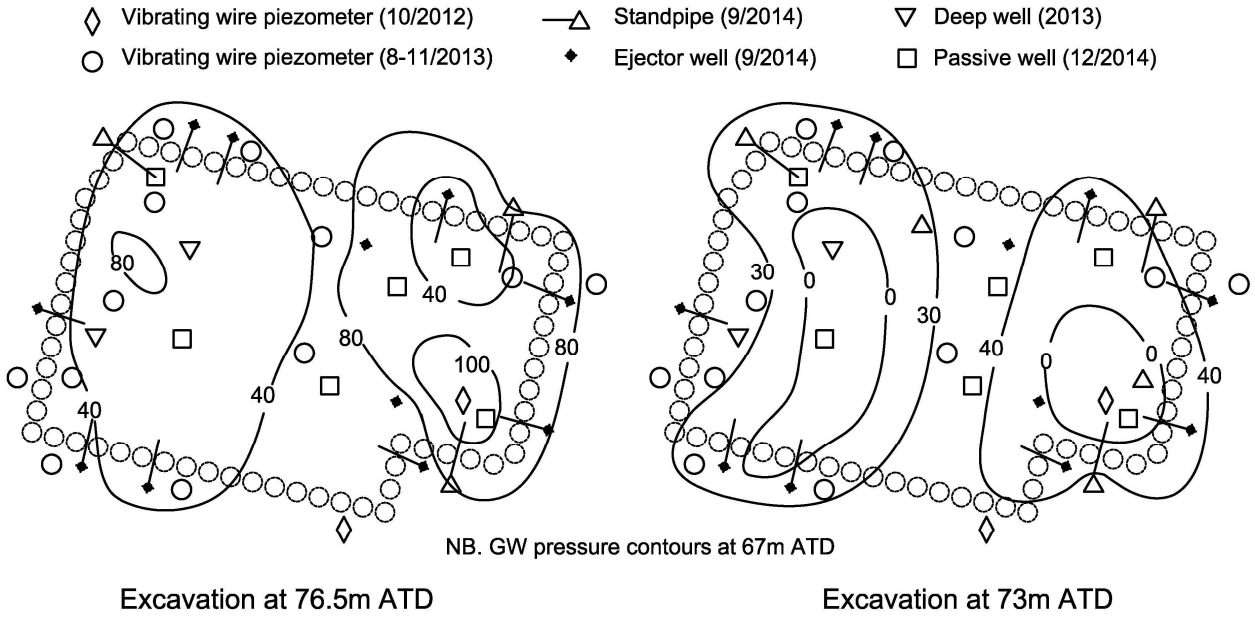


Figure 14. Crossrail Blomfield box, groundwater pressure contours, before and after final modifications to groundwater control systems

This indicated that the sand channel and laminated beds were part of two independent hydrogeological systems. Those within the laminated beds were being recharged (and were probably part of an extensive "sheet" of permeable soil with a much larger footprint than the tunnels and box). In contrast, the upper sand channel seemed to be relatively isolated and once dewatered by the tunnel dewatering it was not vulnerable to groundwater recharge. In the laminated beds, groundwater pressures increased by 30 kPa in 60 days, reaching a value of 115 kPa (when the excavation was approaching 83m ATD).

The ejector wells located inside the box began to reduce groundwater pressures during September and October 2014. In the upper sand channel, groundwater pressures were reduced below zero (with the vibrating wire piezometers recording small negative pressures between 10 and 20 kPa). However, in the laminated beds groundwater pressures could not be reduced to below 75 kPa. Once the excavation had reached 76.5m ATD, the groundwater pressure in the laminated beds was close to the amber trigger level. It became clear that contingency measures would need to be implemented to avoid breaching the red trigger at formation level (of 25 kPa, i.e. one third of measured values).

In addition to the different behaviour between the laminated beds and upper sand channel, there was also different hydrogeological behaviour between the western and eastern halves of the box, Figure 14. Groundwater pressures below the eastern area were significantly larger than those beneath the west and it was considered that this asymmetric behaviour was due to the presence of faulting in the Lambeth Group across the south-eastern part of the box. It was considered that another six wells would be needed to reduce pressures in the laminated beds. The practicality of installing six wells was a concern given the confined conditions at that level inside the box.

Installation of all the wells in a single stage was likely to seriously delay the main excavation works, so the decision was made to split the excavation area into two zones, to allow the first three wells to be installed in the eastern zone whilst allowing excavation in the western zone. This staged approach allowed well installation and operation to be carried out in parallel with the excavation. When the excavation approached 73m ATD, an area of "wet" very silty sand was identified across the northern half of the box, with stiff clay to the south, Figure 15. Narrow slit trenches were installed in the norther half to 70m ATD with local sump pumps. Once activated these further reduced groundwater pressures to below the amber level within the Lambeth Group and enabled the final formation level of 71m ATD to be safely reached, Figure 14. Once the excavation had reached 71m ATD a 300mm thick blinding slab was cast (this acted as a temporary base strut) to support the box during the Christmas holiday period. Passive pressure relief wells were extended above the blinding to relieve any groundwater pressure below the blinding slab. The final layout of deep wells, ejector wells, piezometers and standpipes are shown in Figure 16.

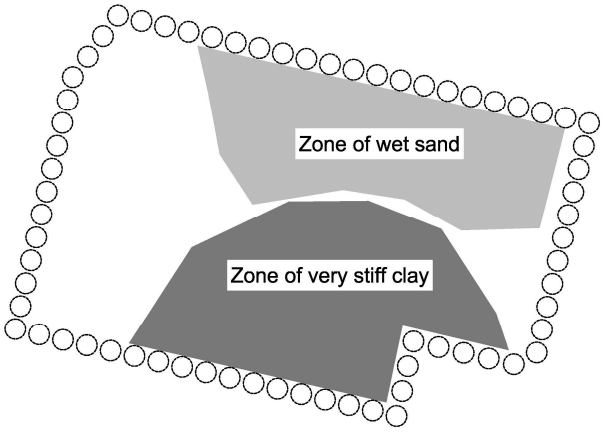


Figure 15. Crossrail Blomfield box, configuration of clay and sand layers at 73 m ATD

4.2.7 *Summary*

The OM, using progressive modification, enabled the dewatering to be successfully implemented, despite significant geological variability, and provided assurance of site safety and also enabled considerable time and cost savings to be achieved.

In total nine ejector wells and twelve deep pumped wells were used, compared with the original base case layout of 22 ejector wells and 20 deep wells, i.e. about 50% reduction due to the application of the OM. In part this was achieved because only the laminated beds were subject to groundwater recharge via a remote source, and also due to each installed well having a wider zone of influence than originally assumed. The position of each well could also be optimised relative to observed effectiveness (which was partly influenced by the fault zone). The complexity of the macro-fabric of the Lambeth Group was revealed by careful logging of the excavated material and from logging of all the drilling for instrument and well installations, Figure 17. The complex inter-layering of the sand, silt and clay layers, and local faulting could not have been predicted from any ground investigation; as noted in the introduction, these conditions are an example of Terzaghi's *"minor geological details that are unknown"*. Hence, accurate predictions of dewatering requirements could not practically have been made at the design stage.

The use of the OM by progressive modification is ideally suited to the implementation of dewatering, especially within the complex geology encountered at the Blomfield Box. The 42m deep box penetrated into the Lambeth Group which comprised a complex interbedded sequence of water bearing strata, with groundwater pressures of up to 250 kPa. Within the congested confines of the site, ground investigations were severely constrained, and even if full ground investigations could have been carried out there would have been considerable residual uncertainty. An optimistic dewatering design based directly on the predictions of the most probable conditions would have been unsafe. In practice, observed conditions proved worse than these conditions. An overly conservative design based on the worst credible conditions would have been impractical to install in a single phase. The observed groundwater pressures during the box excavation varied in a highly complex manner, both spatially and with time, as summarised in Figures 13 and 14.

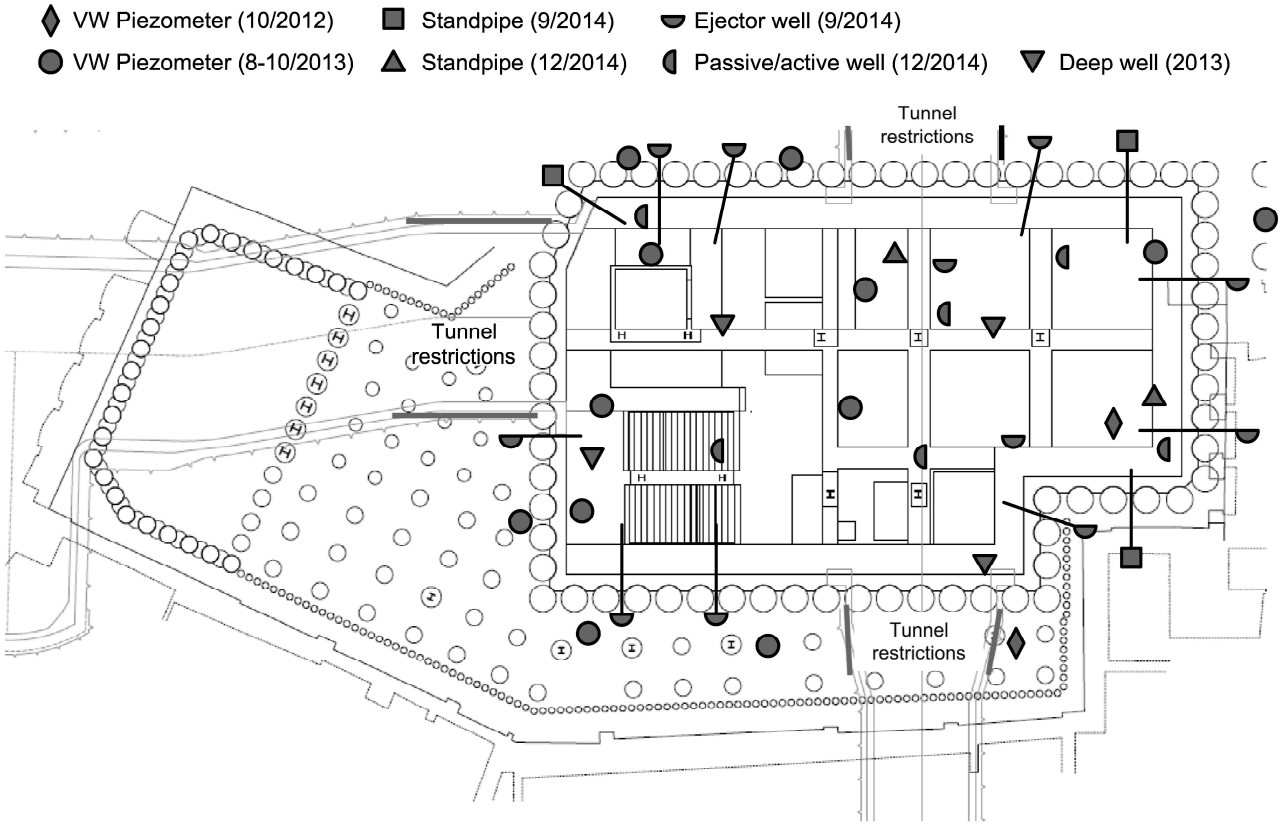


Figure 16. Crossrail Blomfield box, final layout of groundwater monitoring and wells (note slit trenches not shown for clarity).

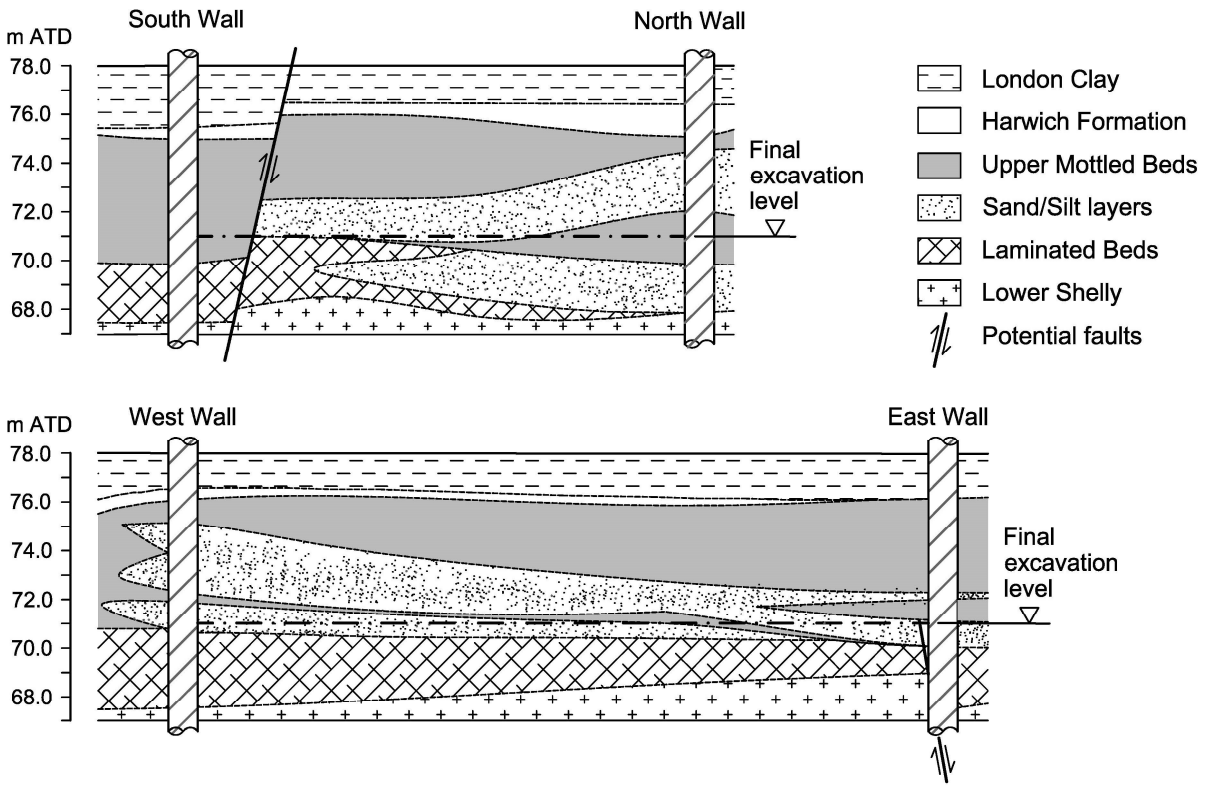


Figure 17. Crossrail Blomfield box – logged profile through Lambeth Group.

4.3 *Case History 3, HS2 propped retaining walls– early application of RTBA*

4.3.1 *Introduction*

This case history summarises recent experience gained on a section of the HS2 project located in the west midlands, UK (Liew et al, 2026). It focuses on the retaining walls constructed for the approaches to Bromford Tunnel, which is a 5.6 km twin-bore high-speed rail tunnel connecting Water Orton in North Warwickshire with Washwood Heath in Birmingham, UK. The Bromford Tunnel East Portal (BTEP), located near Water Orton, serves as the launch site for the TBMs, whilst the Bromford Tunnel West Portal (BTWP) at Washwood Heath functions as the reception point. The timely completion of these portal structures was essential to prevent delays in the HS2 tunnelling programme.

4.3.2 *Key aspects of design and construction*

The BTEP is an underground box, 80m long, 33m wide, and up to 16m deep (Figure 18). The BTWP is slightly smaller, 60m long, 33m wide, and up to 22m deep (Figure 19). Importantly there is a sump structure, up to 7m deep, located near the centre of each portal, which substantially increases the effective depth of each retained structure, to up to 23m and 29m for BTEP and BTWP respectively.

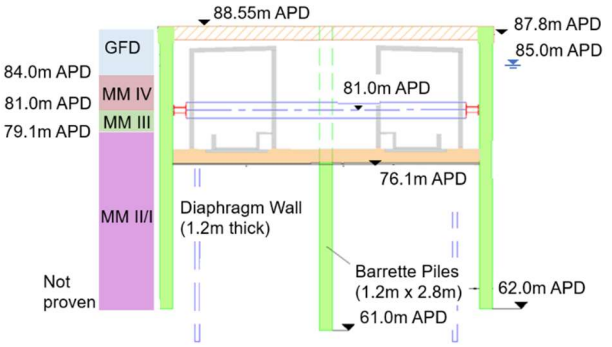


Figure 18. Typical section at BTEP (Eastern End).

The WHRC is a 1.3km open-cut structure connecting to the BTWP at its eastern end and transitioning to embankments near Washwood Heath Depot in the west. The Washwood Heath Retained Cut, WHRC, represents one of the most substantial structures of its type along the route (Figure 20). It is approximately 1.2 km in length and has a maximum retained height of nearly 20m. Along the length of the WHRC the support system varies as the retained height reduces, as indicated in Table 10. The retaining walls were generally constructed using a 1.2 m thick diaphragm wall, except for the westernmost 200m of the WHRC which used 1.2m diameter secant piles. The portal and retained cut structures were designed for top-down construction. Before excavation, abstraction wells were installed to lower the groundwater level inside the structures. In the original design, the temporary steel props could only be removed after the base slab was cast and the concrete reached the required minimum strength. For the portal structures, this would be followed by the construction of the sump structure.

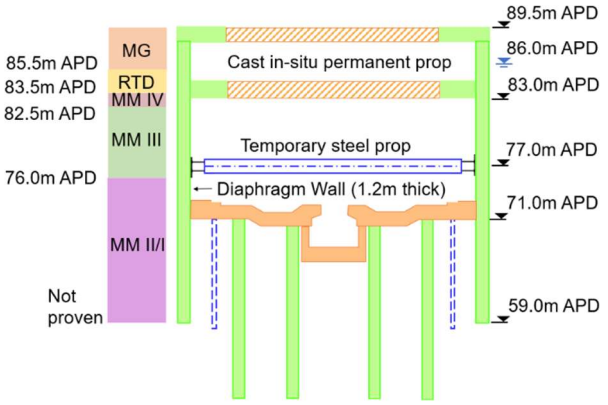


Figure 19. Typical section at BTWP.

Table 10. Washwood Heath Retained Cut – support system**

Section	Length (m)	Max. Depth* (m)	Support system
A	145	19	Two-level permanent props: pre-cast and in-situ beams; one-level temporary props
B	185	16.5	One-level permanent props; one-level temporary props
C	290	13	One-level permanent props
D	125	10.5	One-level temporary props
E	200	7.5	One-level temporary props

*Depth includes haunches, **Original design basis, without OM.

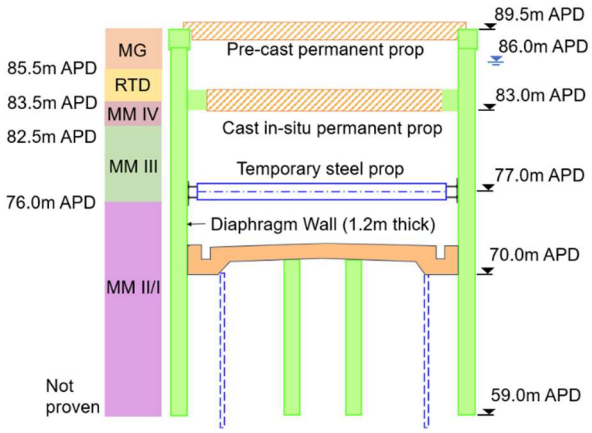


Figure 20. Typical section at WHRC (Section A).

4.3.3 *Ground and groundwater conditions*

Along this section of HS2 the predominant geology is Mercia Mudstone, which can be characterized as an Intermediate Geomaterial (IGM), typically varying from a stiff clay to a weak rock. The weathered 'soil-like' layers are usually characterized as low-plasticity clayey silts or silty clays. The overlying superficial deposits at the BTEP are Glaciofluvial Deposits, whilst Made Ground, Alluvium and River Terrace Deposits overlie the MMG along the BTWP and WHRC.

The challenges typically associated with an IGM are discussed, for example, in FHWA 1999. Mercia Mudstone (previously known as Keuper Marl) is a well-known deposit (CIRIA C570) from a general geological perspective, but there is no precedent experience in Mercia Mudstone of deep retained excavations at the scale envisioned for HS2. Table 11 summarises typical descriptions of the MMG based on borehole data. Though the description of Mercia Mudstone Grade III is "soil-like", the material exhibits strength characteristics that are fluctuating between very stiff clay and very weak rock. The

Grade II/I mudstone typically had an unconfined compressive strength, UCS, of about 2 MN/m2. The MMG, in general, is intrinsically difficult to characterise due to its high vulnerability to sampling disturbance, which means laboratory derived strength/stiffness data is rarely representative of in-situ mass behaviour. In-situ testing is also challenging due to varying bands of hard layers interbedded with softer deposits, together with groundwater seepages leading to local softening; hence, even pressuremeter testing gives inconsistent indications of strength and stiffness. The rock mass quality is highly variable both due to several faulted zones being traversed (the precise geometry and orientation of these faults could not be practically determined) and due to variable dip and dip direction of joints within the rock mass. The mass permeability is also variable, being mainly influenced by the nature of the rock joints, (varying from high permeability where joints are open to low permeability where joints are closed). Hence, it was considered likely that earth and groundwater pressures during construction could vary significantly along the retaining walls, as conditions varied from worst credible to most probable. Groundwater level is approximately 85m APD at the BTEP, and 86m APD at the BTWP and WHRC (Figure 18, Figure 19, Figure 20). The groundwater pressure increases hydrostatically with depth.

Table 11. Description of the Mercia Mudstone Group

Geology	Typical description
Mercia Mudstone Grade IV	Firm to very stiff slightly sandy reddish brown, mottled greenish, grey clay. Sand is fine to coarse. Occasional presence of mudstone lithorelicts up to 25mm.
Mercia Mudstone Grade III	Stiff to very stiff gravelly clay. Frequent presence of lithorelicts greater than 25 mm. This layer is likely to be interbedded with extremely weak rock that has been softened and altered during the drilling process.
Mercia Mudstone Grade II/I	Very weak to weak mudstone with subordinate siltstone and sandstone. Mudstones can exhibit a 'blocky' structure. Interlaminated mudstones and siltstones occur within the formation.

4.3.4 *Achieving agreement to use the OM*

The original design support system provided a conservative design basis, it also introduced temporary works with significant cost, programme, and carbon implications. Opportunities were explored to create a more efficient construction process by implementing the Observational Method (OM) through Progressive Modification. It was anticipated (as noted above) that for most of the retaining walls the actual earth pressures during construction would probably be less onerous than assumed for design. The use of the OM would also mitigate risks if worst credible conditions were encountered in local areas, for example due to adversely oriented faulting.

However, achieving agreement to use the OM was a time-consuming process, and was particularly challenging due to the complex approval, checking and assurance processes specified for the project. A key first step to achieving agreement was the implementation of a technology trial on the western end of BTEP in 2021(which was constructed in accordance with the original design). The technology trial was for use of RTBA software (DAARWIN, De Santos 2015) which uses machine learning-based algorithms to enable near real-time back analysis (RTBA). Liew et al. (2023) discuss the trial and concluded that the back-analysed stiffness for the MMG was significantly higher than indicated by the available ground investigation data and assumed for the original design. The success of this trial facilitated agreement with all the project stakeholders to use the OM for the eastern end of BTEP, BTWP and, most significantly, the adjacent WHRC. An important element of the approval to use OM was an optimised workflow to expedite the OM checking and assurance process (and the inclusion of an independent checker with OM experience). The primary aim of using OM was to eliminate the temporary steel props to speed up the construction programme. Additionally, the aim was to remove the "sequence constraint" by completing excavation before installing permanent props along the WHRC, i.e. the walls would temporarily act as cantilever rather than propped walls.

4.3.5 *Application of the OM*

One of the initial OM tasks was to review and modify the originally specified instrumentation and monitoring plan to ensure it was suitable for the OM. OM instrumentation comprised primary and secondary instruments. The purpose of the primary instruments was to control the construction process, whilst the secondary instruments provided additional insight into the ground-structure interaction behaviour and allowed for cross-checking of the primary instruments.

The primary instruments were, for the initial excavation stages only, in-place inclinometers (IPIs) and Shape Accel Arrays (SAAs) embedded in the diaphragm wall, with automatic readings taken every hour. Empty inclinometer tubes were also installed adjacent to each inclinometer to provide redundancy in the monitoring scheme. Optical displacement sensors (ODSs) were installed on the inner surface of the exposed walls to monitor wall convergence. Importantly the ODSs, which require minimum interpretation and enable rapid decision-making, replaced IPIs / SAAs as the primary system once the excavation reaches the level where the ODSs are installed (Liew et al. 2024). The secondary instruments included subsurface instrumentation for ground deformation and groundwater pressure.

The performance limits set for the OM were bounded, at the red limit, by structural design considerations (specifically, the need to control crack width for the permanent reinforced concrete diaphragm walls) and, at the amber trigger, by pragmatic assessments of monitoring accuracy and the need to avoid frequent breaches of the amber trigger (i.e. the amber trigger needed to be larger than anticipated wall movements for most probable conditions) whilst maintaining adequate band width before the red trigger was reached. The red trigger had to be sufficiently below the red limit to ensure adequate time was available for the blinding strut contingency to be cast (based on site specific logistics). The maximum crack width for the diaphragm walls was a project specified serviceability limit state, SLS, for these structures, to ensure long-term durability requirements were satisfied. Fundamentally this relates to wall curvature, but for practical OM implementation was simplified to a maximum wall displacement (which varied for the different structures, based on retained height and structural support conditions). Eadington et al (2024) discuss the factors which influence the flexural stiffness of reinforced concrete retaining walls, and their detailed back-analyses demonstrate that actual flexural wall stiffness can deviate from commonly assumed values. These structural factors are important when considering structural demands on underground structures and especially when design is controlled by structural serviceability criterion, such as crack width. The project's crack width criteria are onerous which led to onerous wall displacement limits. The red trigger values were approximately 0.1% of retained height for the portal structures and approximately 0.15% of retained height for the WHRC structures, which are similar to commonly observed wall displacements for embedded retaining walls in stiff ground (e.g., Long, 2001). In the authors' judgement, given that the MMG was likely to be more competent than the 'stiff ground' in Long's database there was

a high probability that wall displacements would typically be far less than 0.1% of retained height for the HS2 structures (consistent with the, albeit limited, case history data for retaining walls in mudstones, e.g. Fernie & Suckling, 1996). Nevertheless, these relatively small limiting wall displacements demanded a rigorous and accurate instrumentation and monitoring system.

To manage the observational procedures and fulfil project assurance requirements, Verification Points (VPs) were introduced (Liew et al. 2016). The following activities were undertaken when a VP was reached: a detailed review of instrumentation and monitoring data was conducted. The numerical model was calibrated against selected and processed measured wall displacement data. Calculations of wall displacements for subsequent excavation stages were updated. The potential for making beneficial changes to the design and construction was assessed, as well as the potential to implement pre-planned contingency measures. A meeting with the client HS2, contractor BBV and the independent checker was held to agree to the next steps. The pre-agreed contingency measure, if a red trigger was breached, was a 300mm thick structural concrete blinding strut (C35/45 concrete mix). This would be implemented within 24 hours of reaching the excavation level. The concrete would cover the entire plan area, be flat and level, and have clean contact with the wall.

The intent was to use RTBA to support OM implementation and this represented the first large scale use of RTBA for OM in the UK. However, experience gained during early testing of RTBA and for the BTEP trial highlighted two significant issues:

(a) The RTBA algorithm calculates a mathematical optimum for a parameter, such as a variation of ground stiffness with depth, but this mathematical optimum may not be physically reasonable from a geotechnical perspective. Detailed interrogation of the analyses often revealed more reasonable parameters which were just beyond the mathematical optimum. This emphasized the critical role of expert judgement during RTBA applications;

(b) A critical enabler of RTBA is a robust, structured data management system that ensures timely access to carefully checked, selected and reliable monitoring data. This required a 'filter' between the site monitoring system and data entered into the RTBA. For the HS2 project, Mott MacDonald developed a bespoke solution designed to manage the vast volumes of data generated by the project's extensive geotechnical monitoring network

Hence, to complement the RTBA platform, a centralised Instrumentation and Monitoring (I&M) repository has been developed, as discussed by Wride et al (2026). The workflow begins with the ingestion of data from a variety of sources - such as sensors, instrumentation and monitoring data, and site reports (e.g., excavation progress and staging data). These inputs are automatically processed through a suite of cloud-based ETL (Extract, Transform, Load) Python scripts, which clean, standardise, and upload the data into the central repository. An additional set of integration scripts facilitates communication between the repository and the RTBA. This integration allows engineers to readily compare and contrast the results of back analyses, original design predictions, field monitoring data and various other data related to the project (such as temperature and excavation levels data). This streamlined workflow has significantly reduced the time previously spent on data processing, thereby allowing the site team to focus more effectively on interpretation, engineering judgement, and risk-informed decision-making.

4.3.6 *Field observations*

The OM was first applied at the eastern end of BTEP, and the initial measured wall displacements were sufficiently small for agreement to be reached at VP1 for the temporary steel props to be omitted. As excavation progressed from VP1 to VP2, confidence grew that the amber trigger would not be breached. This enabled the construction of a trench up to 2.5m deep below the base slab for the installation of a carrier pipe before casting the portal base slab, further accelerating the construction programme. Maximum observed wall displacement was close to two-thirds of the 10mm amber trigger value (at VP2), significantly lower than the red trigger at 15mm. The most influential parameter affecting wall displacement is the mobilized stiffness of the MMG, Figure 21 compares the back-analysed shear modulus profile at BTEP with that assumed from the available ground investigation data for the original design. At depth within the MMG the back-analysed shear moduli were typically more than double the original design values. It is notable that the mobilized ground stiffness exhibited a rapid increase of stiffness with depth, whereas the original design profile (based on correlations with unconfined compression strength, UCS, data) assumed a near constant value. The mobilized shear moduli were about 80% to 90% of Go (from downhole geophysics, measuring vertical shear modulus).

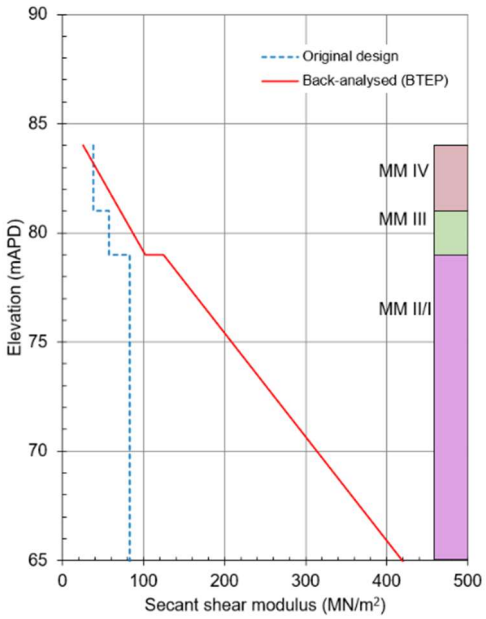


Figure 21. Comparison of secant shear modulus values of the Mercia Mudstone Group at BTEP

For BTWP, Figure 22 shows the measured and RTBA calculated horizontal wall displacements across VPs for both total cumulative displacements and incremental displacements for specific construction stages. The IPI, ML169_IPI602, was selected for the back-analysis due to its good repeatability and consistency, further supported by its strong correlation with the corresponding ODS 0004 data. The calculated horizontal wall displacements were derived from the RTBA software, commencing with the back-analysed parameters from the BTEP. At each VP, RTBA recalibrated the numerical model using live data from the selected IPI, enabling forward calculations of wall displacements, which were then checked against the pre-defined trigger levels. This feedback loop enabled the rapid assessment of excavation performance, validate model calculations, and determine whether temporary support was required at each VP. For BTWP the back-analysed

stiffness values of the MMG were generally within 15 percent of those at BTEP. Figure 22 also shows that the wall displacements were less than the pre-defined amber trigger levels at each VP. The maximum measured wall displacement was about 10mm at VP5, compared to 15mm and 23mm for the amber and red trigger levels, respectively. Given the good match between the measured and RTBA calculated values, the good reliability of the instruments, and the significant margin between the measured and trigger values, Mott MacDonald proposed proceeding with the construction without any temporary support at each VP. This decision was made during the weekly CTC (Contract Technical Committee) meetings attended by BBV, HS2 and the independent checker. The agreement to proceed with the excavation was sought and signed by the relevant parties as part of the assurance process. Table 12 outlines the BTWP and WHRC construction sequences.

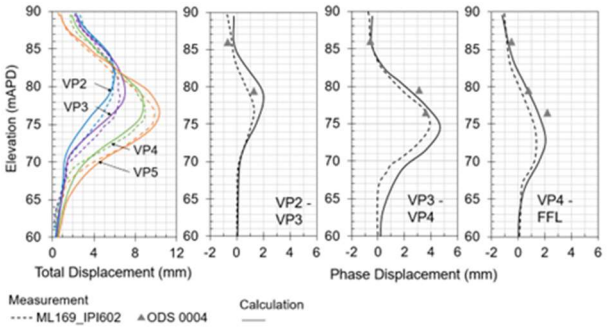


Figure 22. Measurement vs RTBA calculation – total and phase displacement at BTWP.

Implementation of the OM at WHRC commenced after BTWP and the confidence gained at BTEP and BTWP enabled a more efficient construction sequence to be adopted for the WHRC. The decision was made to delay the installation of the precast top prop for section A, Figure 20. The RTBA derived parameters from BTWP indicated that there was a high probability that during initial stages of excavation the wall could act as a cantilevered section whilst remaining well within its SLS structural capacity. This delay of top prop installation significantly eased site access, removing headroom constraints and enabling more efficient plant movement and excavation operations within the retained cut. Furthermore, without headroom constraints, safety risks across the site area, (due to prop installation and subsequent prop strike risks), were substantially reduced for this stage of construction (Figure 23). Excavation toward VP1 (84.8mAPD) advanced through Made Ground and River Terrace Deposits under cantilever wall conditions. This stage was characterised by predominantly stable displacement trends once excavation level was reached, remaining well below amber trigger values. Typical displacements for the north wall of WHRC are shown in Figure 24. These show maximum cantilever wall displacements of the order of 10mm, and during final excavation stages, with a top prop, maximum wall displacements are less than 15mm. Obviously across the length of WHRC wall displacements varied, but were generally within +/- 30% of the values shown in Figure 24.



Figure 23. A view from BTWP towards WHRC, August 2023

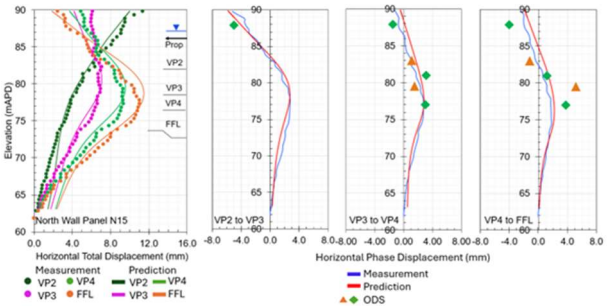


Figure 24. Results of near real time back analysis of WHRC, section A, North wall Panel N15 (ML1244-SAA802).

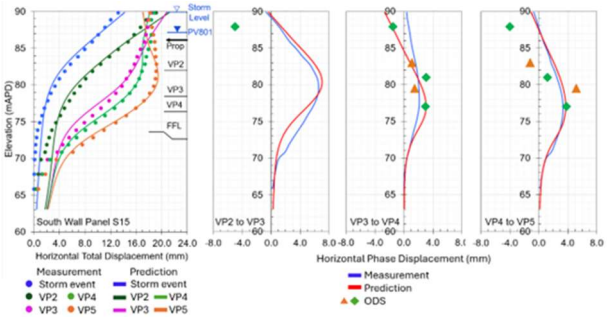


Figure 25. Results of near real time back analysis of WHRC, section A, South wall Panel S15 (ML1244-IPI801).

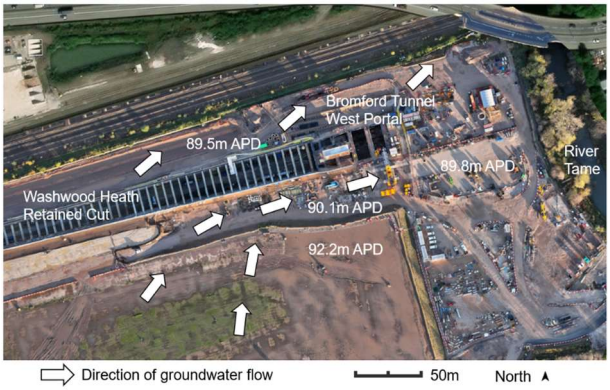


Figure 26. Direction of groundwater flow at BTWP and WHRC

Table 12. Summary of construction sequence at BTWP and WHRC Sections A and B.

Construction Activity	Bromford Tunnel West Portal			Washwood Heath Retained Cut – Section A			Washwood Heath Retained Cut – Section B		
	Start Date	End Date	Duration (Days)	Start Date	End Date	Duration (Days)	Start Date	End Date	Duration (Days)
Excavation to VP1	07/07/2023	31/08/2023	55	26/06/2023	13/10/2023	109	04/10/2023	11/07/2024	281
Cast intermediate prop	22/09/2023	01/12/2023	70	08/12/2023	07/02/2024	61	22/03/2024	13/05/2024	52
Excavation to VP2	23/11/2023	24/01/2024	62	19/01/2024	23/04/2024	95	23/04/2024	30/08/2024	129
Excavation to VP3	12/01/2024	26/02/2024	45	01/03/2024	23/05/2024	83	13/05/2024	06/09/2024	116
Excavation to VP4	07/02/2024	06/03/2024	28	11/03/2024	09/07/2024	120	20/06/2024	05/09/2024	77
Excavation to FFL*	14/03/2024	09/04/2024	26	23/04/2024	27/06/2024	65	11/07/2024	14/10/2024	95
Install precast top prop	—	—	—	03/10/2024	09/10/2024	6	—	—	—
Cast base slab	30/05/2024	24/07/2024	55	08/10/2024	07/05/2025	211	11/11/2024	16/05/2025	186

Note: FFL is final formation level or base of excavation

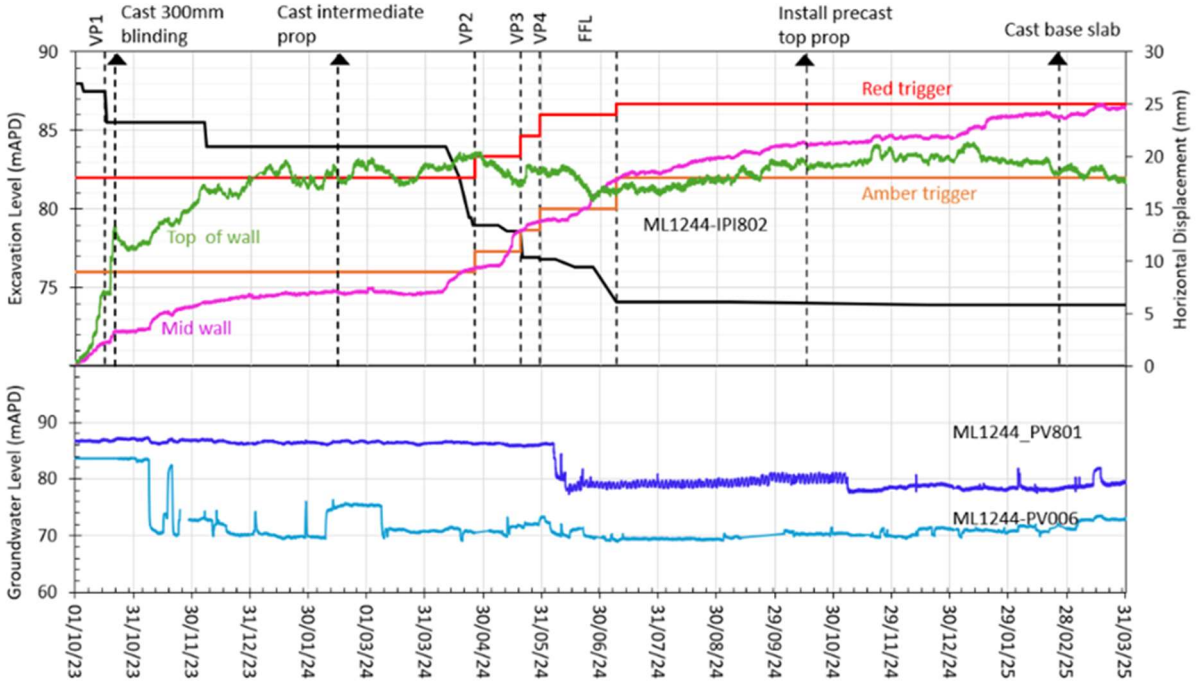


Figure 27. Progression of wall displacement and groundwater level over time at WHRC, section A, panel S15.

A significant exception to the wall displacements shown in Figure 27 occurred in a 20m section of the south wall of the WHRC, at panel 15, and these locally much larger movements were triggered by an extreme weather event. The contrast in wall displacements is shown through comparison of Figure 24 with Figure 25. On 17 October 2023, excavation activities were impacted by the occurrence of Storm Babet, which brought prolonged extreme rainfall across the West Midlands. This effect was further exacerbated by the direction of groundwater flow, which is predominantly to the northeast towards the River Tame, Figure 26. Notably, after the installation of the south wall, the flow direction on the southern side shifts eastward due to the wall locally acting as a dam. In contrast, groundwater on the north side continues to flow northeast, away from the wall. The replacement of the Made Ground and Alluvium layers behind the south wall with a combination of well-graded granular fill (from ground level to 88.5m APD) and cement-stabilised granular fill (from 88.5m to 84.5m APD) may also have altered the hydrogeological regime.

The event resulted in temporary flooding and ponding of groundwater locally behind the south wall (panel 15) of the retained cut which led to a rapid acceleration in wall movement in this locality with the maximum measured wall displacement exceeding the amber trigger levels, Figure 27. A risk reduction meeting was held between BBV, Mott MacDonald and the independent checker. Because of the rapid acceleration of wall movement at S15, the decision was made to implement the red trigger contingency measure (even though the red trigger had not been breached at this stage). Hence, a 300mm thick structural concrete blinding strut was cast across the width of the excavation at S15 (a 10m length of the retained excavation). This contingency measure substantially slowed the rate of wall movement and assured the safety of the subsequent works. As excavation progressed towards the base of excavation level (FFL), mid-wall displacement at S15 increased, Figure 27. The mid-wall displacement rate reduced after groundwater control measures lowered the groundwater level behind the wall, indicated by ML1244_PV801 in Figure 27, and after top prop installation. However, during the 9 months between reaching the FFL and casting the base slab, time dependent maximum wall displacement gradually increased (a creep rate of less than one mm per month) and approached the red trigger level of 25mm. The wall displacement stabilised after the base slab was cast and achieved its design strength.

Figure 28 shows the back-analysed parameters for the BTWP and the WHRC south wall at panel 15. As expected, there is a decrease in the mobilised stiffness of the MMG III when compared to the back-analysed stiffness at BTWP, despite the two structures being adjacent to each other. Normalised small strain stiffness (G/G0) for this location at WHRC was determined to be 50% to 70% of G_0, compared to 80% to 90% of G_0 at BTWP. This is consistent with the lower wall system stiffness (Long et al, 2001) for a cantilever wall at WHRC versus the multi-propped wall at BTWP, and also the likely adverse effects of the adverse weather event which affected this part of the WHRC.

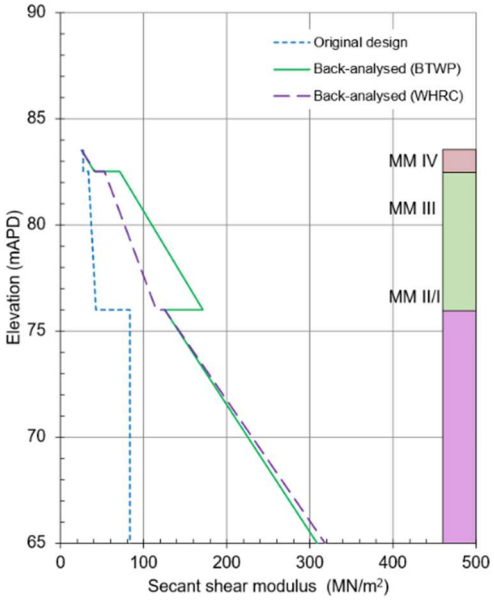


Figure 28. Comparison of secant shear modulus values of the Mercia Mudstone Group at BTWP and WHRC (panel S15).

4.3.7 *Summary*

Despite the challenges described above, the close collaboration among all parties enabled the ultimate goals of the OM to be achieved: no additional temporary support was required (with the exception of the local area near panel S15 discussed above), and the time constraint for the installation of top precast props was successfully removed. Installing the precast props at a later stage brought many benefits, including improved construction flexibility, enhanced site access by eliminating headroom constraints (making it easier for machinery and personnel to move within the retained cut and BTWP), and increased operational efficiency with fewer physical obstructions.

Overall, the implementation of the OM for the HS2 retained excavations at BTEP, BTWP and WHRC was judged to be a considerable success. The time saving allowed the construction team to better prepare for the launch of the first TBM and its subsequent removal. The omission of dozens of temporary steel props resulted in substantial benefits. These included significant savings in the construction programme of the order of several months, a reduction in carbon emissions of approximately 3,000 to 4,000 tonnes CO_2e and enhanced safety achieved by eliminating restrictive temporary works. For IGM such as the Mercia Mudstone, which are intrinsically difficult to characterise through conventional field and laboratory testing, the OM enabled the geological and hydrogeological uncertainties to be safely and cost-effectively managed. This included safely managing the local effects of an extreme weather event, which led to temporary flooding and impounding of groundwater behind a section of the retaining wall.

The back-analysis confirmed that the mobilised field stiffness of the Mercia Mudstone increases rapidly with depth, which contrasted with the available UCS data for the mudstone at depth. RTBA, via the process summarised in 4.3.5 above, proved invaluable in characterising the mobilized stiffness of the Mercia Mudstone.

5 LESSONS LEARNED

5.1 *Instrumentation and Monitoring Systems*

In recent years there have been impressive improvements in instrumentation and monitoring, I&M, technology. The technical capability of modern instrumentation is more than adequate for practical engineering purposes, provided it is properly installed and calibrated. For example, in the UK the Cambridge Centre for Smart Infrastructure and Construction (CSIC) at Cambridge University has promoted the innovative use of emerging technologies in real-time sensors and data management, Mair (2016). These new systems can provide extensive real-time data and have the potential to provide important new insights into ground and ground-structure interaction behaviour. Soga et al., (2015) described the results of fibre-optic monitoring which provided considerable detail of construction induced effects. A key question is: do commonly used commercial procurement methods provide an appropriate environment for reliable I&M data to be obtained on a consistent basis? This is a crucial issue when the OM is to be implemented.

I&M is commonly specified for projects, unfortunately, the purpose is often unclear. In general, I&M is normally specified either as a protective measure for pseudo-legal reasons to confirm that existing infrastructure is not being damaged, or designers use I&M to confirm that their design is adequate. If there are no obvious problems identified then the I&M data is not scrutinised, this defensive or negative intent often means that little value is obtained from I&M. This negative context has a corrosive impact on I&M procurement and data reliability. If, as is often the case, the main contractor employs the I&M specialist(s) to install and monitor instrumentation, they will usually have no obvious motivation to ensure that the I&M is of high quality. In the absence of OM being implemented, the I&M data will not offer any value to the main contractor (other than to indicate that there is a "problem"). Given this context, it should not be a surprise that the quality of I&M data is often poor. The procurement of I&M is usually assumed to be a trade activity and is often let as sub-contracts on a lowest price basis, which regularly leads to disappointment in terms of data quality and reliability. Similar to the challenges faced when trying to implement the OM, the key challenges are contractual, commercial and cultural in terms of how the industry typically operates.

At the current state of practice, basic errors in specifying instrumentation still occur and instrumentation is often installed too late (e.g., refer to Dunnicliff, 1988). If reliable baseline readings cannot be obtained, then diagnosis and correction of instrument and monitoring data may be fatally compromised.

The following issues need to be addressed if the industry is to maximise the value from I&M, and are particularly important when the OM is to be used:

(a) It needs to be recognised that an instrumentation system needs to be designed. The design needs to be based on a sound understanding of the likely deformation mechanisms, relevant ground-structure interaction behaviours, and construction methods and sequencing for the works under consideration. Hence, the instrumentation design should be led by a senior design professional with the requisite expertise.

(b) The advantages and limitations of different instrument types for the specific function required need to be taken into account. Given the rapid changes in instrumentation technology, it would be prudent for the designer responsible to get advice from an instrumentation specialist.

(c) The procurement process for the I&M systems needs careful thought. It is unwise to treat this as a trade activity and for it to be purchased on a lowest cost basis. Table 13 provides a checklist of questions that need careful thought prior to finalising contract documents. I&M should be procured under a professional services contract on a best value basis (considering specific technical proposals, staff capability, as well as price).

(d) As noted above, instruments must be installed early enough to ensure adequate baseline readings are obtained, so seasonal / weather related trends are understood, and potential problems can be diagnosed and resolved prior to construction commencing. This could be achieved, at least for some instruments, if they were installed through an early works contract (by the client) or during stage 1 of an ECI contract. This should certainly be implemented if sensitive infrastructure needs to be monitored.

(e) A question which is not easy to resolve is: who is the optimum employer of instrumentation contractors? Is it the client or the main contractor? There are advantages and disadvantages for each scenario, as noted in Table 13. Perhaps the optimum approach is for the client to act as employer during an early-stage contract, as noted in (d) above, then for the contractor to act as employer during construction. If OM is being implemented, then the main contractor will be highly motivated to ensure the I&M is reliable. For major projects, an independent specialist could be employed, on a professional services basis, to audit the instrumentation contractors' installation methods and system functionality, during the early stages of a project.

(f) Responsibility for the quality of I&M needs to be assigned to senior professionals with relevant skills and authority. Firstly, the I&M contractor should be responsible for installation, operation and maintenance of the instrumentation systems. Secondly, once construction commences, a senior engineer for the main contractor (with sufficient competence and authority to resolve problems quickly) needs to be appointed to co-ordinate the I&M with construction activities and resolve access issues. For example, during construction, it may be necessary to temporarily halt construction to resolve I&M problems or instigate replacement of damaged equipment, and this is only likely to occur if the I&M engineer has a senior role in the construction site management team.

(g) The factual I&M data needs to be accessible to all key project stakeholders on a near-real time basis. There are several propriety software systems available which can collate, summarise and plot I&M data with easy access for selected parties. This would often be let as a separate sub-contract to instrument installation, especially for major projects. It is recommended that the employer for the I&M data management system is the client.

Table 13. I&M procurement questions

Key Questions	
WHY	Instrumentation is needed – state specific purpose for each instrument at a designed location. Focus on quality NOT quantity.
WHO	is responsible for quality of instrumentation - single professional with adequate competence + authority to resolve issues should be appointed. is under CONTRACT with who (no panacea): • If I&M employed by Client – then potential access difficulties in active work areas? Who is responsible if instruments faulty? • If I&M employed by Main Contractor – often insufficient time for baseline readings? Is there motivation to obtain high quality data? Ideally, two phase approach: • 1st phase I&M employed by client; • 2nd phase employed by Main Contractor. Procurement on professional service / quality basis.
WHEN	will I&M be installed - need to assess background / environmental effects, and obtain reliable baseline monitoring before construction starts.
DETAIL (Specifications)	Specifications – avoid 'cut + paste' (specialist input on the latest technology is needed)! Prepare performance-based specifications: focus on required outcomes, organisation, competency, quality/reliability checks, diagnosis + resolution of problems, data management + information flows + reporting.

5.2 *Importance of Simplicity and Effective Communication*

It is essential that the practical implementation of the OM (such as contingency measures, monitoring, trigger limits) is kept as simple as possible. This is necessary both to maximise the likelihood of achieving agreement to use the OM and to ensure the OM is effectively implemented. Although much sophisticated and creative thinking is often required by an OM Champion, simplicity both in communication and execution is central to the OM. The importance of this was articulated by Peck (2001): "*the OM paid off handsomely, without more than the most elemental theory and with only qualitative predictions. There were few refinements, and no elaborate computer modelling to be 'validated' by exotic remote-reading sensors. These refinements are not the essence of the OM. They have their place, but they should not deflect attention and resources from the essence of the method*". Therefore, the temptation to over-complicate the method should be resisted. The OM provides a simple way to address and resolve complexity. All the case histories discussed by Powderham and O'Brien (2020) involved the resolution of very complex conditions. For example, the Boston Central Artery Tunnel Jacking, (Powderham and O'Brien, 2020), involved a sensitive network of seven interconnecting railway tracks which was subject to seven different sources of ground movement. These included tunnel jacking, jet grouting and artificial ground freezing within deep deposits of made ground and alluvium which had been affected by two centuries of waterfront development. This complexity was resolved on a simple basis through one set of OM control traffic lights focused on the allowable rail track movement limits. A risk for OM implementation can be generated by a project's political environment. This may lead to the proliferation of non-critical instrumentation and the imposition of supplementary management systems (and perhaps several 'traffic lights') by different stakeholders - together with overly prescriptive requirements. Such complications are likely to substantially compromise the effectiveness of the OM.

Specific roles and responsibilities for OM implementation should be clearly established and documented, for example Table 14. The owners of the various tasks would usually include representatives from the designer and contractor (and when appropriate the client) working collaboratively within a single project 'OM team'. The frequency of monitoring review meetings must be defined (e.g. daily or weekly during critical phases as appropriate). Actions should be defined if trends demonstrate opportunities for beneficial changes or if adverse trends develop, the implementation of corrective contingency plans. Construction control flow charts should be produced, examples are given in Figures 29 and 30, to ensure key data and site observations are recorded on a regular, unambiguous, planned and consistent basis.

Table 14. Example of roles and responsibilities during the OM.

Team Member	Role	Responsibility
Designer	Engineering consultant (OM designer)	Inspect bridge and monitor condition. Check and interpret instrumentation. Provide technical advice to Railtrack during the OM implementation.
Railtrack	Project Manager and Track infrastructure owner	Safe operation of railway. Manage works and instruct Contractor. Communications with P-Way contractor.
Preload Contractor	Build pre-load embankment (OM contractor)	Safe implementation of works. Carry out mitigation measures as instructed.
Instrumentation Contractor	Install instruments, provide factual data	Check instruments functioning correctly. Replace faulty instruments, report factual data to OM designer, contractor and Railtrack.
Permanent Way Contractor	Track maintenance	Maintain track alignment within Railtrack prescribed tolerances for twist.

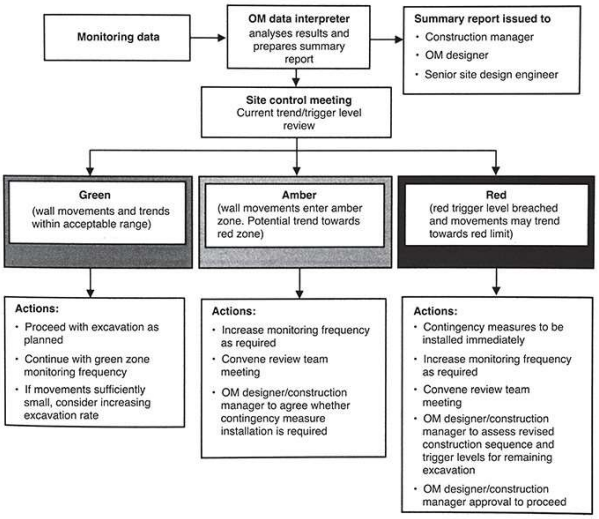


Figure 29. Example of OM site control flow chart (after Powderham and O'Brien, 2020).

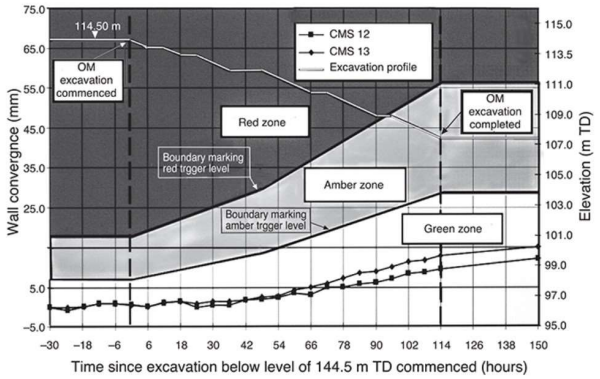


Figure 30. Example of OM traffic light control zones (after Powderham and O'Brien, 2020)

5.3 *Contingency measures*

The identification, design and planning associated with contingency measures are crucial tasks. Sometimes these are conventional solutions, such as inserting additional temporary supports for an excavation, or simply allowing more time for consolidation of an embankment built on clay soils (as for case histories 3 and 1 respectively discussed above). Appropriate materials and equipment have to be mobilised to site, so the contingency measure can be installed within an agreed timescale if a trigger limit is reached. The appropriate timescale requires consideration of the time required to obtain and interpret the critical observations compared with the timescale for deformation to reach a critical magnitude. Contingency planning requires close collaboration between designer and contractor and, often, creative lateral thinking to solve specific project constraints and the key is usually simplicity and speed of installation. The identification of a simple and rapidly installed contingency measure can be the crucial factor in being able to implement the OM. This creates opportunities for creative collaboration between designer and contractor. Examples which unlocked the use of OM include (as discussed by Powderham and O'Brien, 2020):

- For Heathrow airport airside road tunnel, the use of blinding struts as a potential contingency measure rather than conventional props
- For Wembley Stadium the identification of mass concrete counter-weights as a contingency (to mitigate excessive pile-group rotation)

5.4 *The Power of Progressive Modification*

OM implementation through Progressive Modification, (Powderham, 1998), is when construction is started by implementing a design with a level of risk judged acceptable by all parties. This may vary from 'worst credible' through to 'most probable' conditions, although commonly the assumed starting conditions are more cautious than most probable. Monitoring is then used to enable modification of the design, on a step-by-step basis, as construction proceeds. Hence, progressive modification allows for the gradual introduction of changes, based on observed behaviour, to move towards actual conditions. The implementation of the OM through progressive modification has been found to be the best basis for achieving agreement with, typically skeptical, project stakeholders. With its additional focus on enhancing and demonstrating safety, progressive modification provides greater comfort to the stakeholders in their concerns about risk.

Progressive modification can address both main categories of OM, Figure 31. There is a big difference commercially and politically between 'best way out' situations, as opposed to 'ab initio' ones. It is essential to appreciate that each application of the OM is bespoke, and beyond the broad concepts, each case needs to be addressed in detail for the particular project conditions. In this context, progressive modification brings very effective flexibility. Consequently, there is no need to commence construction with an unduly pessimistic design based on worst credible conditions or one based, with little safety margins, on estimations of the most probable conditions. Instead, the conservatism of the starting condition can be chosen to accommodate the comfort levels of the stakeholders for each specific case and reflect specific project constraints and risks / opportunities.

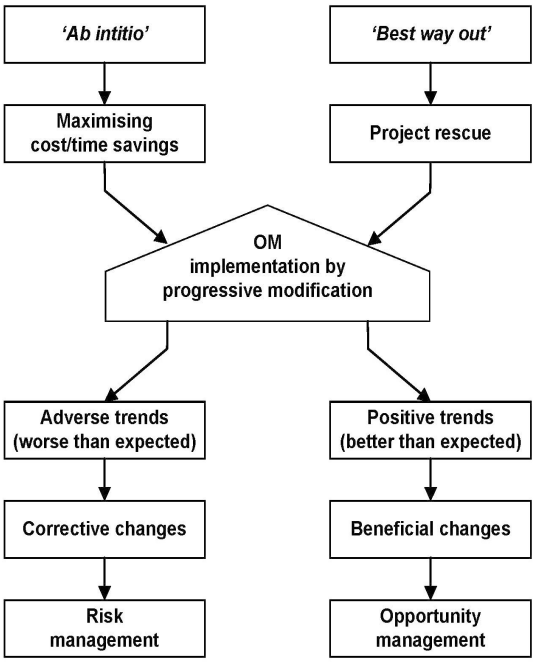


Figure 31. Progressive Modification, all OM categories can be addressed.

It is vital to remember that uncertainties are not solely related to ground parameters. In any Civil Engineering endeavor, particularly involving underground construction, there can be many uncertainties, including those associated with: structural aspects (especially for existing infrastructure), ground-structure interaction, construction processes, ground treatment, etc and issues which can be categorised as "construction workmanship". These various "uncertainty components" are not necessarily independent and are often interactive with other components, Figure 32.

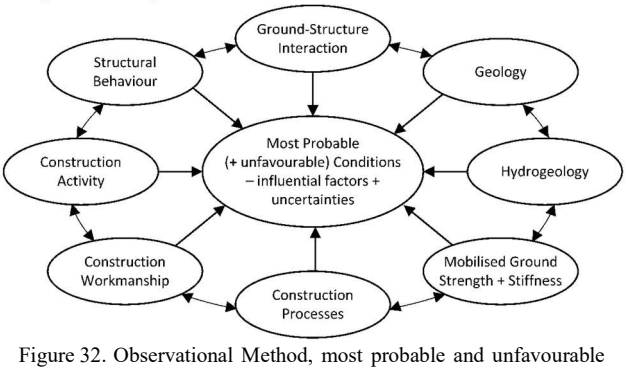


Figure 32. Observational Method, most probable and unfavourable conditions - some influential factors.

These range of uncertainties all need to be assessed, noting that some aspects may not be amenable to analysis. The importance of considering a wide range of uncertainties was summarised in Peck's eight steps (Peck, 1969), in particular the second step: "*assessment of the most probable conditions and the most*

unfavourable conceivable deviations from these conditions." In any OM application robust risk management and maintaining and demonstrating an acceptable level of safety is crucial, also refer to O'Brien et al (2026b). In the author's experience commencing the OM from "most probable" conditions is generally unacceptable to one or more of the key project stakeholders. Furthermore, as noted by Muir-Wood (2000), this might lead to a more expensive outcome. If the OM is implemented through Progressive Modification, then there is no need to commence construction with an unduly optimistic set of assumptions. Progressive Modification does not need to be constrained by any prescriptive requirement to implement and complete an application of the OM with a fixed level of conservatism. It is essential, however, to have a realistic appreciation of the potential range of conditions from the most probable to the worst credible, together with practical plans for how the associated risks and opportunities can be managed. Such an understanding is central to establishing the viability of any application of the OM. This assessment requires, as well as investigation and analysis, experience and engineering judgement. It is important to note that design modifications do not have to be contingent measures of a corrective kind, although this is often assumed. It is implied, for example, in the relevant clauses relating to the OM in Eurocode 7. Rather than design modifications being restricted to corrective contingencies, progressive modification inherently offers the potential for a fully flexible process. Through its step-by-step approach, it creates the opportunity to maximise value by facilitating the introduction of beneficial design changes in a safe and controlled manner. And, if the OM is commenced from a demonstrably safe conservative base, most of the design changes are likely to be beneficial. Moreover, this managed incremental approach also facilitates a very effective way of dealing with unfavourable conditions (for example, refer to case history 3 above). Thus, with this ability to satisfy safety concerns while also maximising value, progressive modification creates a fertile basis on which to achieve agreement to implement the OM.

5.5 *Setting Performance Limits*

An important practical development was the use of a simple traffic light system, (Powderham, 1998), to define performance limits for critical observations, Figure 33. Establishing these limits is a good example of where the OM is not well served by a prescriptive approach. Powderham and O'Brien (2020), outline ten different factors, summarised in Table 15, most of which will be site and structure specific, which should be taken into account when determining performance limits. Establishing a performance limit, or trigger level, may be relatively straightforward if the critical factor is solely associated with ground failure. However, failure mechanisms involving ground-structure interaction are more common and generally more complex, involving a wider range of factors. This is especially the case when the OM is used in urban areas, and the performance limits are required for protection of existing infrastructure. For these situations, the critical performance limit typically relates to allowable movement of an existing structure, ie SLS considerations. As noted by Burland (2024) the allowable movement of structures is very subjective and varies both with the function of the structure and the reactions of users (and their technical advisers). Any rational assessment of allowable movements should be based on structural capacity, which in general requires an analysis of induced curvature and/or strain within the structure, rather than some arbitrary total movement limit. Examples, for masonry structures, include the charts by Boscardin and Cording (1989) and Burland and Wroth (1974). Although, as noted by Cording et al (2010), structural details, such as wall to floor connection details can dramatically affect how a structure will respond to ground movements (for example, if lateral ground strains will be transferred into a building or not).

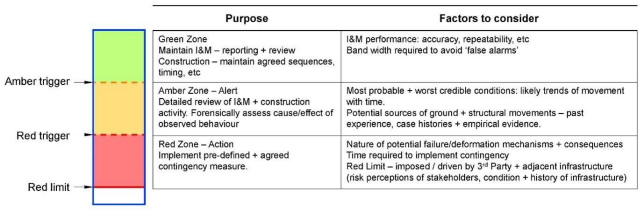


Figure 33. Performance Limits, traffic light system for OM

Table 15. Setting performance limits, key factors

Factor	Considerations
Nature of failure/deformation mechanisms	Does failure or critical deformation involve ground only or ground/structure interaction
Consequences of potential failure/deformation mechanisms	Is the limit set by SLS or ULS, how serious are consequences. Are permanent or temporary works affected or existing infrastructure
Timescale for failure/deformation to reach critical level	This will typically define the margin between red limit + red trigger (based on practical logistics of implementing contingency)
Accuracy, precision + repeatability of instrumentation systems	Key factor in setting the green limit
Is monitoring data relative or absolute	For example, is differential settlement or angular distortion the critical observation
Nature of critical observation, ease of collation + interpretation	How much data manipulation is necessary, ideally critical observation is kept as simple as possible
For most probable conditions what is likely trend + will it remain in green zone	Ideally this should be based on reliable + relevant case history data, ie avoid solely relying on analysis.
If worst credible conditions - will trends develop towards red trigger levels	Would these trends allow enough time to implement planned contingencies
Is there enough margin in the green, amber (if used) + red zones	It is important to avoid false and/or frequent breaches of trigger levels. Speed of contingency installation, critical for setting red trigger (relative to red limit).
Can critical observations be captured within a single set of performance limits.	In general, OM should be managed through a single set of performance limits.

Table 15 highlights why performance limits need to be site and structure specific, and why geotechnical and structural engineers need to work closely together to set appropriate limits. A common issue is the assessment of existing infrastructure (eg buildings, tunnels, underground utilities, rail systems), potentially affected by new construction. Their structural condition and history are often poorly understood and may have undergone several modifications/repairs in the past.

A classic example, of the challenges associated with this type of assessment is given by the Mansion House case history (Powderham and O'Brien, 2020), an 18th Century building of national importance. One of many issues, was the degree of building stiffness which could be relied upon, and how the greenfield settlement trough (induced by tunnel construction) would be modified by any building stiffness. In addition, an adjacent large building also interacted with the Mansion House, hence any settlement trough was a combined function of tunnelling works, plus interaction between the two existing buildings. Given the different stakeholders involved, achieving a consensus on the 'most probable' conditions for this situation was impractical (even unlimited analytical power would not resolve this type of issue). For this case history, performance limits were set on worst credible assumptions (of negligible building stiffness), and OM through progressive modification incrementally demonstrated building stiffness was significant in this case, and the tunnelling settlement trough was considerably different and less onerous than greenfield assumptions.

Performance limits should be developed on a pragmatic basis, and for some OM applications the three zones, red/amber/green traffic lights, have been changed to two zones only. Powderham and O'Brien (2020) and Pye et al (2022) give examples of how OM has been safely implemented with two zones (red/green only). The original basis for an 'amber' zone was to enable more focused interrogation and analysis of instruments which were showing significant changes. When the traffic light system was first developed, some three decades ago, there were greater data management challenges and instrumentation accuracy was more limited than is currently the case with modern systems. More recently, for applications of the OM on relatively small geographically constrained sites, the performance limits have been set on a simple red/green basis. The greater accuracy of modern instrumentation (when well installed), together with modern data management systems, means there is less risk of data overload. Back-analysis has then been used to assess the risk of the red trigger being breached in subsequent construction stages, Pye et al (2022). Nevertheless, for very large sites and where ground/groundwater conditions are highly variable the original three zone traffic light system remains a valuable and effective way to focus attention on more critical areas of a site.

5.6 *Limitations of Analytical Predictions*

Although analytical capability has increased dramatically through the widespread availability of advanced numerical modelling, this has not led to consistent improvements in performance predictions of geotechnical structures. For example, a recent comprehensive review (Gaba et al, 2017) of retaining wall and associated ground movements concluded that '*ground movements cannot be predicted exactly'*. There was considerable scatter of observed maximum retaining wall movements for similar wall types in similar ground conditions. The scatter was attributed to a range of construction and groundwater related issues, including: local variations in construction sequence, timing of wall support installation, construction details and local effects of groundwater seepages and associated time-dependent changes in ground strength. These issues, because of their sensitivity to as-built construction sequences and timing, are not amenable to prediction by a designer prior to construction. Obviously, these intrinsic challenges are exacerbated if ground investigation data or engineering expertise is inadequate for the problem under consideration

Ground movement predictions in urban areas are particularly important, due to the need for minimizing the risk of damage to existing infrastructure. However, the reliability of predictions in these situations are often poor due to the legacy of historic construction activities, and the need to remove 'obstructions' prior to commencing the proposed permanent works. For example, for the Boston Central Artery project (Powderham and O'Brien, 2020) it was necessary to remove numerous timber piles and other obstructions associated with two centuries of waterfront development, Figure 34. The ground movements associated with the removal of man-made obstructions would not normally be considered amenable to analytical predictions. Typically, analytical predictions will focus on a few activities for forming permanent works, such as retaining wall deflection and bulk excavation. Whereas, observations have shown that large ground movements can arise due to miscellaneous construction and temporary works activities which are not within the scope of designer's analytical predictions, such as: underpinning of adjacent buildings, placement and removal of temporary access ramps/earthworks, pile installation, local ground improvement (such as grouting), for example refer to Korff and Mair (2013); Fernie et al (2001). For these situations, the OM would be the most practical means of controlling construction to minimise the risk of damage to existing infrastructure.

The OM is a means of effectively closing the gap between observed behaviour and that which may be anticipated on the basis of conventional design calculations. This gap between theory and reality can often be very large indeed. Six decades ago, Terzaghi (1961) reflecting on this gap wrote:

"Base the design on whatever information can be secured. Make a detailed inventory of all the possible differences between reality and assumptions. Then, compute on the basis of original assumptions, various quantities that can be measured in the field. On the basis of the results of such measurements, gradually close the gaps in knowledge and, if necessary, modify the design during construction."



Figure 34. Removal of timber piles and other obstructions, after Powderham and O'Brien 2020.

The iterative application and recalibration of numerical modelling during OM, for example as discussed by O'Brien and Liew (2018) is an application of Terzaghi's advice. Recent OM applications have used Bayesian updating (e.g. Lacasse and Di Biagio, 2019; Wu, 2011). Tschuchnigg et al (2026) provide an introduction to RTBA, and discuss its potential and challenges. The application of advanced numerical modelling in geotechnics has been discussed by O'Brien and Higgins (2020) and they have proposed a framework for practical applications known as the 'ten steps' – refer to Table 16. The application of

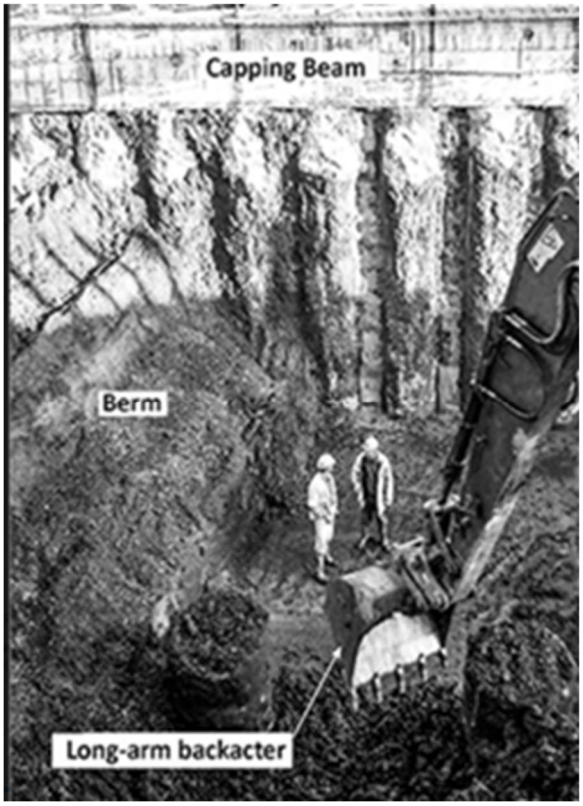
the 'ten steps' is also relevant for OM applications and RTBA to ensure the outputs from numerical models are robust and reliable for the specific application.

Care is required when output from a back-analysis is re-used to predict behaviour, either for subsequent construction stages or for apparently similar design problems in similar geology. Figures 35 and 36 illustrate a few examples of the factors that need to be considered. Construction activities are invariably three-dimensional, 3D, and these 3D effects vary during construction, either reducing or increasing depending upon the nature of the works being built, Figure 35. These 3D effects are exacerbated by ground-structure interaction and by non-linear time-dependent ground behaviour, Figure 36. Therefore, a 2D back-analysis may provide a pessimistic or optimistic value of, say, ground stiffness, if re-used for subsequent analysis depending on the differences in geometry between the back-analysed problem and the issue being evaluated. The phenomenon of ground arching, Figure 36(a) is a ubiquitous aspect of ground-structure interaction, as noted by Terzaghi et al. (2009). Common situations, such as ground improvement (using stiff ground reinforcing elements) or propped excavations, are significantly affected by arching. Earth pressures are locally modified, with the load following the stiffest load path (e.g., 'active' earth pressure reduces on a wall spanning between stiff props and the prop load increases proportionally).

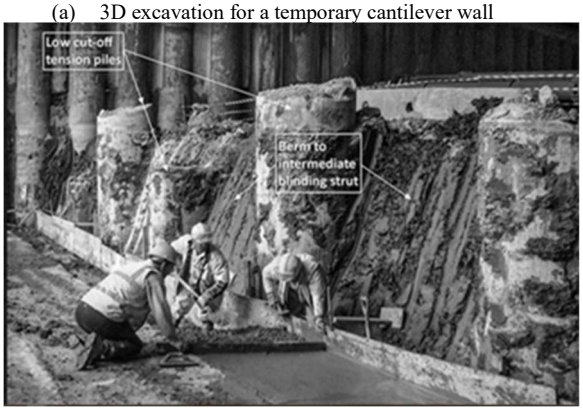
Step	Description	Comment
1	Objectives	Specify why modelling is needed, and what outputs are critical for design. Discuss with experienced modeller. Assess options and limitations.
2	Application Category	May vary from simple modelling through to complex modelling which is safety critical. This will affect the necessary level of checking and seniority of staff involved. Past experience in type of modelling and specific application being considered is an important factor.
3	Design Team Organisation	Roles and responsibilities. Is the modeller also the designer? Information flow, etc. Who checks/approves? The time and budget available are also key factors.
4	Competencies and Training	Necessary skills/experience? Future training needs?
5	Information for Model	Surveys, Ground Investigations, Design/Site Information, Construction Scenarios, etc. Adequate for scope/complexity of modelling?
6	Simplifications and Assumptions	Some idealisations are always necessary, and need to be stated clearly. ALL assumptions and data sources must be described, communicated/agreed across project team, client, etc.
7	Calibrations / Testing	What types of calibration/testing will be made, e.g. element, component or case history checks. Important for assessment of model output reliability
8	Robustness	Some sensitivity studies are necessary to check for "cliff-edge" effects.
9	Checking	Checks for both Inputs and Outputs, independent of originator.
10	Reporting	Broken down into three parts: assumptions; factual reporting of the analysis; and finally interpretation of output, (often this will be the most complex task!)

Table 16 The Ten steps for advanced numerical modelling, after O'Brien and Higgins (2020)

Arching mechanisms are particularly important for ground-structure interaction because of the ground's highly non-linear and hysteretic stiffness response, Figure 36 (b). If a simplistic linear-elastic Mohr-Coulomb soil model is used to back analyse an early construction stage (when mobilised ground strains are small) and this back-analysed stiffness is used for a later stage (when mobilised ground strains will inevitably increase), it would likely lead to unsafe performance predictions. These errors may be compounded if time-dependent changes in ground strength/stiffness become significant. Simplistic soil models can be used for back-analysis, but the engineer needs to make cautious allowances for changes in ground behaviour, which are difficult to judge, either for subsequent construction stages or re-use in future design studies. In general, it is more appropriate to use an advanced non-linear model which can account for relevant aspects of ground behaviour, such as stiffness non-linearity or coupled-flow.

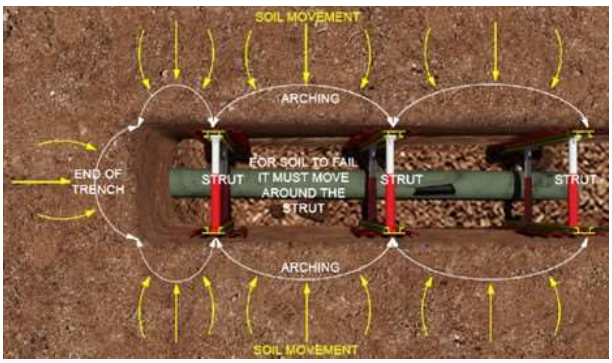


(a) 3D excavation for a temporary cantilever wall

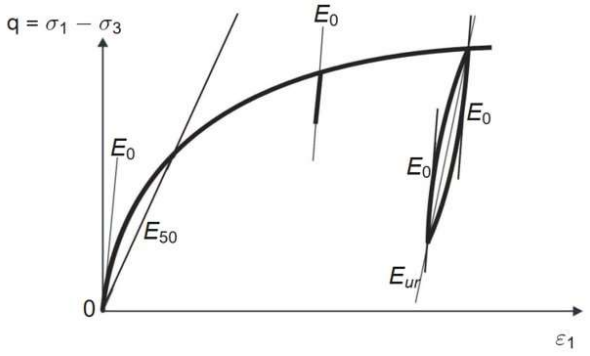


(b) complex 3D geometry, retaining walls, tension piles,etc

Figure 35. Potential back-analysis pitfalls, evolving 3D geometry during construction



(a) Arching mechanisms



(b) Non-linear and hysteretic ground behaviour

Figure 36. Potential back-analysis pitfalls, taking account of arching mechanisms and non-linear ground behaviour.

6 FUTURE POTENTIAL

The OM has delivered, as discussed in this paper, significant benefits on the projects where it has been used. Modern technology, in combination with OM, could amplify these benefits beyond specific OM applications. In recent years there have been considerable technological developments in the following areas:

- Modern real-time instrumentation
- Monitoring data-base systems
- Real Time Back Analysis, RTBA, techniques

RTBA has the potential to deliver efficient back analysis of geotechnical structures and therefore become an important tool to support future OM applications. This is particularly the case for large projects, which generate huge volumes of monitoring data and traverse a wide range of ground conditions and structural configurations, as discussed in case history 3 in 4.3 above. RTBA enables the rapid recalibration of numerical models by systematically comparing model outputs with actual field data, adjust input parameters based on discrepancies between predicted and observed behaviour, and then most importantly can store the resulting analysis outputs in a readily accessible format.

If the above technologies are integrated through the use of the OM, then there is the basis for a step change in geotechnics design practice and an improved understanding of ground-structure interaction; provided that the technologies are used wisely. A particular advantage of modern information technology (supported by careful use of artificial intelligence techniques, such as machine learning) is that it is possible to create an 'OM data platform' which hosts all the relevant key data (e.g., geology, hydrogeology, structures, construction and selected I&M data, etc), as discussed by Wride et al (2026). Hence, an accessible 'OM data platform' has the potential to be immensely powerful for knowledge management and enabling lessons learnt from past projects to be transferred into beneficial re-use for future design and pro-active asset management. Historically, this has been a particular problem for the construction industry, as noted in several industry reviews, such as the UK Government's Construction 2025 (2013), which noted that *"learning from construction projects is generally lost once a project team disbands at the end of a project"* and *"the lack of integration both between design and construction and construction management and its execution leads to lost opportunities for innovation"*. These problems could be resolved through wider use of 'OM data platforms.' Through using modern technology, selected reliable high-quality data and analysis outputs could be stored within an accessible data repository. The concept and benefits of "Digital Twins", refer to: digitaltwinhub.co.uk, and Bolton et al (2018) especially for more proactive and cost-effective asset management, has been a high profile topic in recent years. Currently, Digital Twins are usually BIM models which, although containing a lot of information, are passive and unintelligent. The incorporation of an 'OM data platform' into a Digital Twin would greatly enhance its capability and provide a better basis for more informed decision-making during the operational life of an asset, as indicated in Figure 37.

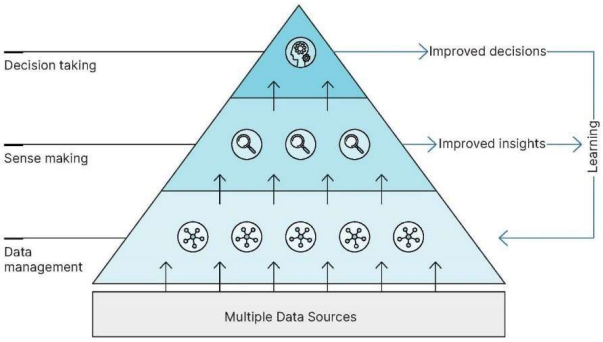


Figure 37. OM enhanced Digital Twins, moving from data to decision making and improved outcomes.

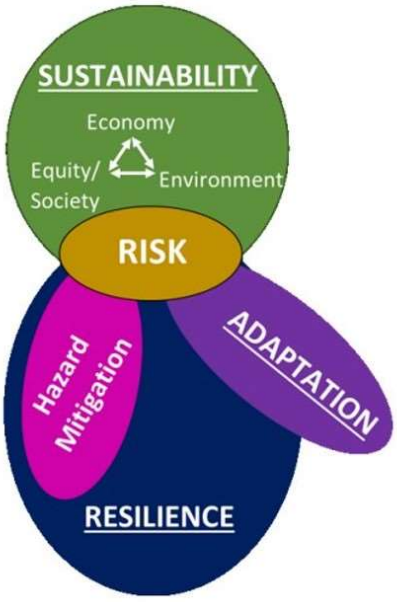


Figure 38. Adaptive design – balancing risk, resilience and sustainability

A key challenge in the design of new civil engineering infrastructure or the maintenance of existing infrastructure is to balance carbon consumption whilst also providing suitable levels of asset resilience in response to the increasing demands of evolving climate change. This balance could be met by applying 'Adaptive Design' principles (for example, refer to ASCE 140, 2018). Rather than design for extreme (and likely poorly predicted) future design events (associated with changing climate conditions), the intent is to design infrastructure which can be modified in the future, on the basis of monitoring of actual performance of the built infrastructure. Hence, this is applying the principles of the OM from short-term applications (as is typically current practice) into long-term applications through the operational life of the structure. Adaptive design has the potential to minimize capital costs as well as reduce carbon use but requires asset owners to invest in proactive asset management systems. For adaptive design to be viable across large-scale infrastructure networks in the future, extremely large monitoring systems would need to be managed efficiently. Hence, where adaptive design is implemented then the OM combined with modern technology has the potential to play a pivotal role in future infrastructure design and operation.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 *Conclusions*

Terzaghi's warning that there are – '*many situations when the geological conditions preclude the possibility of making an accurate performance forecast',* remains relevant. The three case histories discussed in section 4 of this paper all dealt with geological conditions where there was considerable uncertainty in terms of: the time required for consolidation (case history 1), groundwater seepage and the continuity of permeable soil layers (case history 2), the practical impact of joints / faults within a very weak rock mass (case history 3). As noted by Terzaghi, in these geological conditions, if the OM is not used then the outcomes are either uneconomic designs (based on worst credible assumptions) or unsafe designs, and to ignore these situations represents a misuse of soil mechanics. Hence, there are compelling benefits in the use of the OM, such as cost/time savings, improved safety and risk management, etc, refer to Table 2 for examples. Despite these benefits the OM is significantly under-used. This neglect is typically imposed by contractual conditions (which act as a disincentive to its use) or adverse perceptions of undue risk (in part due to prevalent industry culture). Contractual and commercial issues also adversely affect instrumentation and monitoring, and recommendations for improvements in current procurement practice are discussed in Section 5.1 of this paper. The inclusion of a value engineering, VE, clause in contract conditions, which allows potential savings to be equitably distributed, has often been a catalyst for OM to be used. The OM is also widely misunderstood across the industry. On this matter, OM advocates need to recognize that communicating what OM is and how it can be implemented must focus on the essence of the OM, as recommended by Peck (2001). The OM can be implemented across a wide range of different projects (Table 3).

Agreement to use the OM requires a range of issues (both non-technical and technical) to be resolved. Achieving agreement tends to be simpler if only temporary works is affected by use of the OM and becomes more complex when permanent works or existing infrastructure is affected or if there are complex independent checking or assurance processes. Typically, it requires a determined experienced individual to emerge from a project team, acting as the 'OM champion', to act as an advocate for use of the OM. Gaining agreement requires a flexible approach to address project specific constraints and concerns, however, it is always necessary to:

- Provide a strong business case
- Develop a sound technical basis, (typically this requires structural engineering, as well as geotechnics expertise)
- Produce robust risk management
- Gain the trust of key stakeholders

Technically, the essence of the OM and the key to success in applying the OM is in identifying the critical observations and having the means to obtain and act upon them (ie through pre-planned contingency measures) in a safe and timely manner. This latter aspect relates to a cardinal limitation of the OM: if it is not possible to change the design during construction, then this renders the OM a non-starter. It must also be possible to address the range of credible conditions (both through the observations and the contingency measures), varying from worst credible through to most probable. The OM is most effective where the critical observations can be reliably obtained through simple measurements. Safe and effective implementation of the OM requires close teamwork and effective communications, both across disciplines (eg between structural and geotechnical engineers) and between designer and contractor. One of the main uncertainties which needs to be managed through the OM are time-related effects, since this will influence the viability of the OM. When relevant case history data is lacking, preliminary trials can provide significant benefit.

Peck's eight steps (see Table 1) remain a useful checklist for the OM. However, it is usually inappropriate to commence the OM on the basis of most probable conditions. A more cautious starting point for OM is likely to be safer and more cost-effective; especially if implemented through Progressive Modification, which facilitates a step-by-step evaluation of actual conditions and the introduction of beneficial design changes in a safe and controlled manner. This incremental approach also allows unfavourable conditions to be dealt with.

The multiple stakeholders involved in a project often have widely varying perceptions of risk and how it can be addressed. The implementation of the OM though progressive modification has often been found to be the best basis for gaining agreement to use the OM. With its focus on an incremental, step-by-step, approach progressive modification creates the opportunity to maximise value whilst also allowing OM to start from a demonstrably safe condition. A development of progressive modification, known as the verification process, is facilitating independent design checker approvals within the timescales required by the OM (provided appropriate expertise is actively employed by the checker). Advanced analysis methods play an increasingly important role in this aspect of the OM. However, experienced engineering judgement, based on lessons learnt from relevant case histories, will continue to be central to the safe and effective use of the OM.

The integration of modern instrumentation and monitoring systems and RTBA into readily accessible 'OM data platforms' could form the basis for a step-change in geotechnics practice. It would facilitate more informed decision-making whether for proactive asset management or for new projects. The OM, supported by these technological developments, would also support the use of 'Adaptive Design' principles, which would enable the OM to be used for the design of new infrastructure and facilitate its long-term management and adaption to balance carbon costs and climate resilience.

7.2 *Recommendations for OM Practitioners*

Based on more than three decades of OM experience, the following are recommended:

1. Establish an ongoing site presence and visit the site as early as possible; it is an essential part of observation and assessing site conditions and how these may evolve during construction.
2. Relate design to construction on an informed basis, designers need to have an appreciation of construction plant, means and methods.
3. OM typically requires a multi-disciplinary approach, ensure there is balanced expertise across an OM team (as a minimum this usually requires input from both structural and geotechnical engineers).
4. Recognise that achieving agreement to use the OM often presents a substantial challenge. To reach agreement, the differing commercial issues across the parties need to be addressed, conditions of contract may need modification to facilitate the use of the OM.
5. Treat each application of the OM as bespoke, generally there is a unique combination of skills/attitudes across a project team and its stakeholders, together with project specific constraints, and desired outcomes. Promote teamwork and engender trust. Seek awareness and understanding of stakeholders' priorities and concerns, these will vary between the parties.
6. Keep it simple – but not simplistic. Exercise engineering judgement and avoid undue complication. Advances in technology, for example, in instrumentation and analysis can be a powerful support in the use of the OM, but they are not the essence of the OM. Ensure there is a focus on critical observations and carefully monitor their trends.
7. Study case histories and appreciate the lessons learned.

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