Structural remodelling of existing wall to function as anchored retaining wall – design case study

Belton, G & Merrifield, CM Coffey Geotechnics Ltd, Manchester, UK Garside, N WML Consulting Ltd, Manchester, UK



ABSTRACT

As part of the development of an inner city site including several pre-existing buildings, it was necessary to demolish a three storey municipal building, the rear wall of which was integrated into a reinforced concrete retaining structure. This paper describes processes followed to provide an economic and safe design for the reinforced concrete retaining wall. The iterative nature of the wall support design approach incorporated support from ground anchors; the locations, working loads and inclinations of which were optimized to avoid distress to the wall. Wall displacements were monitored during the demolition of the building, anchor installation and stressing. These show clearly the wall response to removal of support and subsequent application of anchor working loads. Based on the wall response it was concluded that the design approach was appropriate. Rigorous anchor testing demonstrated that the design anchor free and fixed lengths were appropriate.

RÉSUMÉ

Comme la partie du développement d'un site de ville intérieur y compris plusieurs bâtiments préexistants, c'était nécessaire de démolir un trois étage bâtiment municipal, le mur postérieur que dont a été intégré dans un béton renforcé qui retient la structure. Ce papier décrit des processus pour fournir une conception économique et sûre pour le mur de soutènement. La nature itérative de l'approche de conception de soutien de mur incorporée le soutien des ancres de sol; les emplacements, travaillant des chargements et des inclinations qu'a été optimisé dont éviter la détresse au mur. Les déplacements de mur ont été contrôlés pendant la démolition du bâtiment, d'installation d'ancre et d'accentuer. Ceux-ci montrent clairement la réponse du mur à l'enlèvement de soutien et l'application subséquente de chargements de fonctionnement d'ancre. La réponse du mur indique que l'approche de conception était appropriée. L'essai rigoureux d'ancre a démontré que les longueurs libères et fixés étaient approprié.

1. CONTEXT AND BRIEF

As part of the redevelopment of a derelict site in central Huddersfield (West Yorkshire, UK) for the construction of a new further education campus, advance preparatory engineering works were required. The most significant item was to demolish an existing municipal building, West Riding House (WRH), along with other associated smaller buildings obstructing development. The building was situated adjacent to and appeared to form part of a retaining wall supporting a road (Manchester Road) at its crest. The challenge lay in the development of a support system for the wall to replace that provided by the building.

2. OBJECTIVES OF PAPER

The paper presents, as a case study, two contiguous processes: (i) the investigation and analysis undertaken to assess how the retaining wall would behave with the building support removed and (ii) the development of a dynamic design to ensure the long term stability of the free standing wall to retain its function for an extended design life. The main objective of the paper is to present, as a case study, a design process able to respond to the phased discovery of the nature and behavior of an initially propped wall to ensure its long term stability as a free

standing wall and to continue to function as a retaining structure with a live roadway at its crest.

3. EXISTING DATA ON CONDITION OF WALL AND WEST RIDING HOUSE

The original wall was constructed circa 1968 along with the construction of WRH (Arup, 2006). Movement joints were visible within the wall stem immediately to the east and to the west of WRH. Either side of these joints were freestanding cantilever reinforced concrete retaining walls. Figure 1 shows WRH with the retaining wall directly behind it in the background.

From the previous studies and investigations at the site (Arup, 2006 and 2009), there was no historical documented information available on the construction of the wall or of WRH. Limited structural investigations had been carried out previously while the building was still occupied, during which several cores through the floor slab and the walls were drilled (Arup, 2009).



Figure 1. West Riding House.

The initial site investigation for the college development (Soil Mechanics, 2007) attempted to profile the wall. Probes drilled vertically behind the crest of the wall provided inconclusive evidence but did appear to indicate that there was no heel to the wall. Excavations to the front showed that the toe of the freestanding portions of the wall (to the east and west of WRH) extended to 4.2m out from the base of the wall.

Subsequent more detailed investigations were carried out once WRH was vacated (WML, 2009). Trial holes identified that WRH was constructed on the extended base (toe) of the retaining wall and this base projected some 13m beyond the base of the wall. It was necessary to cut back the extent of the toe to allow foundations for the College building to be constructed. The toe was confirmed to be approx 600mm thick with significant depth of lean mix blinding concrete beneath. Cores through the wall identified that the stem was 610mm at Level -2, and 460mm thick at Level -1 and Ground Floor (Figure 2). A typical cross section through the wall and WRH is presented in Figure 2.

Further investigations identified the reinforcement in the upper section of the wall to be 16mm vertical square twist bars at 150mm centres in the far tension face and 10mm mild steel vertical bars at 150mm centres in the near compression face (Figure 3). The equivalent reinforcement in the lower section of the wall was identified as 32mm and 10mm bars respectively (Figure 4). It was concluded that the wall appeared to be acting as a cantilever with horizontal sliding resisted by the mass of WRH.

From inspections of the near face of the wall from within WRH, the wall appeared to be in a reasonable condition, with no obvious cracking or spalling of the concrete or exposed reinforcement. WRH consisted of a reinforced concrete frame construction with internal stability provided by infill brickwork panels. Previous investigations could not identify whether the floor slabs within WRH abutting the wall were acting as props or not.

Reducing the length of the extended toe of the wall and removing WRH would significantly alter the structural behavior of the cantilever wall, leaving it susceptible to failure by sliding, overturning or through inadequate bearing capacity. Any stabilization measures would necessitate the assessment of the current state of the wall and the effect the measures would have on the actions already present. This assessment effectively led to the decision to remodel the wall structurally.



Figure 2. Cross section through wall and WRH (all dimensions in mm).

4. GROUND MODEL AND MATERIAL PARAMETERS

Prior to the design of stabilization for the wall, it was essential to undertake an additional ground investigation. This consisted of two inclined rotary cored boreholes, inclined at 45° to the horizontal, bored to a distance of 25m. These were positioned either side of WRH at a height of 0.5m above ground level. The ground model parameters appropriate for design derived from this investigation are summarized in Tables 1 and 2.

Table 1. Ground model.

Stratum	Description	Upper Level (mOD) ¹
Made Ground	Grey brown clayey angular Gravel with rubble and clay fill	+87.0
Glacial Till	Stiff grey brown sandy gravelly Clay	+78.0
Coal Measures	Weak to strong interbedded Sandstone, Siltstone and Mudstone (thinly laminated, typically very closely spaced fractures at 30-40°)	+75.0

¹ metres above UK Ordnance Datum

Table 2. Material design parameters.

Stratum	γ (kN/m ³) ¹	c' (kPa)	Φ ' (°)	E (MPa) ³
Made Ground	20	0	35	10
Glacial Till	18	0	27 ²	9
Coal Measures	22	720	23	100

¹ unit soil weights derived from BS8002

² angle of shearing resistance derived from equation [1]

³ Young's modulus derived from equation [2]

Given the paucity of information, the following empirical relationships were used to define material parameters as presented in Table 2. The glacial till was assigned a plasticity index (I_p) of 20 for this purpose.

$$\phi_{c}' = 40^{\circ} - 10 \log I_{p}$$
 [1]
(Atkinson, 2007)

E = 0.9 N [2] (*CIRIA*, 1995)

5. DESIGN APPROACH

It was essential that the structural remodeling works did not compromise the integrity and function of the existing retaining wall, during or post-construction. The wall was to be stabilized against overturning and lateral sliding modes of failure, whilst the internal bending and shear stresses experienced by the reinforced concrete sections should at no point exceed their inherent structural capacity. The approach adopted by the authors was to seek a balance of equilibrium such that it was possible to maintain or reduce the bending and shear stresses within the wall and to ensure that tension and compression faces would not be reversed during the remodeling process.

The key elements to the adopted design approach were as follows:

- 1. Establish the basis of design
- 2. Define possible failure modes
- 3. Assess structural capacity of the existing wall
- Investigate limiting equilibrium of the pre-construction state and quantify the stabilizing forces on removal of WRH and cutting back of the base slab
- 5. Using two-dimensional (2D) finite element analysis, model construction phasing and its effect on the wall
- 6. Limit applied anchor forces such that induced actions do not exceed the structural capacity of the wall

5.1 Basis of design

The uncertainty of the degree to which the floor slabs within WRH contributed to the stability of the wall led to the design of a system that would provide lateral support at or close to the point where the Level -1 and Level -2 floor slabs met the wall. A system of two parallel rows of ground anchors installed through the wall would provide such support and was selected as the preferred option to take forward for analysis.

5.2 Failure modes

Once a conceptual design had been selected, the modes of failure associated with removal of WRH and installation and stressing of ground anchors were identified. These included:

- 1. overturning
- 2. lateral sliding
- 3. bearing capacity (beneath base slab)
- 4. bending (of wall)
- 5. shear (within wall)
- 6. punching shear (of anchor heads through wall)
- 5.3 Structural capacity of existing wall

Both upper (460mm thick) and lower (610mm thick) sections of wall were analyzed to determine their capacity in bending and shear. Figures 3 and 4 illustrate the typical cross sections through the upper and lower wall stem, showing the levels of reinforcement in both.







Figure 4. 610mm section (lower stem of wall).

Table 3. Sectional properties and capacities of wall

	460mm	610mm
Concrete characteristic compressive strength, f _{cu} (N/mm ²)	30	30
Steel ultimate yield strength, f_y (N/mm ²)	410	410
Ultimate moment capacity (kNm per metre)	190	896
Ultimate shear capacity (kN per metre)	185	356
Moment capacity controlled by limiting crack width to 0.25mm (kNm per metre) ¹	121	546

¹ Moments expressed as ULS for comparison

A mean concrete compressive strength of 30N/mm² was found from tests on cores taken through the wall stem. Based upon the depth of the sections and implied reinforcement density the bending and shear capacities of the upper and lower wall sections were calculated in accordance with BS5400 Part 4. In addition to calculating the ultimate bending capacities, the maximum permissible bending moments were determined, governed by limiting crack widths to 0.25mm under serviceability limit state (SLS) loading conditions.

Critically, as significant reinforcement had been installed to the far face only, there would be very little ability for the wall to sustain bending which would place the near face in tension.

5.4 Limit equilibrium analysis

Limit equilibrium analyses were carried out to establish the equilibrium of the system under the following conditions:

- 1. existing configuration with WRH in place
- 2. final configuration with WRH removed and anchors installed (for preliminary estimate of anchor loads)

Active earth pressures were assumed to act behind the wall, which along with the 20kPa site load surcharge (based on two portable site cabins stacked one above the other) produced a resultant horizontal thrust of 222kN/m behind the wall. These calculations illustrated that the friction developed at the base of the wall maintained horizontal equilibrium of the wall with a factor of safety of approximately 1.07. The magnitude of the horizontal frictional force at the base is directly proportional to the reaction force developed beneath the base, essentially due to the self-weight of the wall, the building dead weight and the active forces behind the wall, all of which contribute to the overturning moment about the base. For this load condition, the force required to prevent sliding (222kN/m) was less than the limiting frictional force beneath the base (243kN/m), indicating that the wall was stable against sliding in the existing condition, albeit not with a significant factor of safety. Limiting equilibrium was assumed for vertical forces (F_v), horizontal forces (F_h) and moments about the wall toe (M_{O}) .

The load condition derived from the demolition of WRH and the cutting back of the base from 13.1m to 4.5m was also considered with respect to limiting equilibrium. For this load condition the limiting frictional force that could be developed was calculated as 79kN/m (due to the reduction in reaction force normal to the base slab), which is around 33% of the force required to prevent sliding at the base (222kN/m as before). Hence, the wall would be unstable, a mechanism would form and the wall would slide outwards.

This exercise confirmed that the retaining wall would have to be held in position prior to the demolition work commencing. For this purpose, two rows of ground anchors would be installed: one at Level -2 (+80mAOD) and another at Level -1 (+84.25mAOD) with inclinations of 20° and 45° below the horizontal respectively. These inclinations were chosen to achieve a suitable anchorage within rock taking into account the wayleave limits of Manchester Road. From limit equilibrium considerations the top and bottom anchors were to provide pretension loads of 100kN/m and 150kN/m respectively (hence for the proposed 2m spacing of the anchors in the plane of the wall, the pretension force required to be applied in the individual anchors was to be 200kN and 300kN respectively).

Finite element analyses were done in order to gain a more detailed understanding of how the wall interacts with the ground during various stages of construction and to confirm the limit equilibrium considerations.

5.5 Finite element analysis

A two-dimensional plane strain analysis of a representative cross-section was carried out using the finite element program Plaxis (Professional Version 8.6). The staged nature of construction was modelled to determine the stability and deformation of the retaining wall at intermediate steps of the Enabling Works and demolition phases.

The finite element mesh consisted of 15-noded triangular elements for the soil, 5-noded plate elements to model the base and wall, spring elements to model the free length of the ground anchors and tension-only elements to model the grout body at the fixed end of the anchors. The mesh is shown in Figure 5.



Figure 5. 2D finite element mesh.

5.5.1 Initial conditions

The initial stresses were determined via a K_0 -procedure which requires that the soil mass is in equilibrium. Where the ground conditions comprise normally consolidated soils the ratio of horizontal to vertical effective stresses approximates to K_0 , however for this particular load case a more reasonable approximation was likely to be K_a for the soil conditions behind the wall (the limit equilibrium calculations supporting this assumption). The initial pore water pressures were derived on the basis of a ground water table at +73.5mOD.

5.5.2 Properties of the structural elements

An idealized two-dimensional cross section through the wall was analyzed. The parameters assigned to the various structural elements for input into the program are presented in Table 4.

Table 4. Plaxis model input parameters.

Element	Material properties	
Ground anchor, solid bar	$E = 200 \times 10^{6} \text{ kPa}$	
(Szinin diameter)	A = 8.0 x 10^{-4} m ² (per metre run)	
Anchor fixed length grout	E = 10 x 10 ⁶ kPa	
body (72mm diameter)	$A = 4.072 \times 10^{-3} m^2$	
460mm thick wall stem	E = 28 x10 ⁶ kPa	
	v = 0.15	
	$A = 4.6 \times 10^{-1} m^2$ (per metre run)	
	$I = 8.111 \text{ x10}^{-3} \text{ m}^{4}$ (per metre run)	
610mm thick wall stem	E = 28 x10 ⁶ kPa	
	v = 0.15	
	$A = 6.1 \times 10^{-1} m^2$ (per metre run)	
	$I = 1.9 \times 10^{-2} m^4$ (per metre run)	
600mm thick wall base	E = 28 x10 ⁶ kPa	
slab	v = 0.15	
	$A = 6.0 \times 10^{-1} m^2$ (per metre run)	
	$I = 1.8 \times 10^{-2} m^4$ (per metre run)	

5.5.3 Construction stages

The sequence of construction phases modeled in Plaxis is presented in Table 5 below.

Table 5. Intermediate construction phases.

Phase	Description			
0	Generate initial stresses and pore water			
	pressures			
1	Install wall (wished-in-place) with WRH on base			
2	Apply 20kPa surcharge behind wall			
3	Install anchors and zero previous displacements			
4	Pretension anchors			
5	Demolish WRH			
6	Cut back base slab from 13.1m to 4.5m width			
7	Apply horizontal parapet load			

5.5.4 Results of modeling

An assessment of maximum structural action effects induced in the wall at each stage is presented in Table 6.

Table 6. Maximum action effects in wall at each phase

Phase	Bending moment (kNm) ¹		Shear force (kN) ¹	
	460mm	610mm	460mm	610mm
1	38	419	29	171
2	62	546	44	206
3	62	546	44	206
4	128	562	81	193
5	129	530	83	185
6	113	428	79	163
7	116	432	79	164
1	-			

¹ per metre run along wall

The maximum predicted moments and shear forces developed in the wall at all stages are less than the calculated structural capacities of each section presented in Table 3. The final bending moment and shear force distribution within the wall after phase 7 are shown in Figure 6, with wall deflections at phases 4, 5 and 7 presented in Figure 7.



Figure 6. Bending moment and shear force distributions in wall at end of phase 7.

As the anchors were installed at lateral spacings of 2m, a waling beam was designed to span along the upper row of anchor heads. The beam was necessary to ensure that the wall was evenly supported along its length at this level. Only 10mm reinforcing bars were found within the wall and were not sufficient to withstand the horizontal bending moments that would be induced by the wall spanning between anchor locations.

The beam was located below the centroid of the anchor heads inducing a moment in the wall which would act in such a way to relieve the bending moments induced

by earth pressure, thereby effectively strengthening the wall at this critical section.

A waling beam was considered unnecessary for the lower row of anchors as the base slab was considered to act as a stiff horizontal diaphragm, easily capable of transferring loads laterally into the anchors without generating excessive horizontal bending stresses within the wall stem.



Figure 7. Predicted horizontal wall deflections during construction.

6. ANCHOR CONSTRUCTION, INSTALLATION, VERIFICATION AND TESTING

6.1 Construction and installation

There were a number of constraints governing the anchor installation operations within the building, which required careful consideration. A number of walls key to the stability of the building could not be removed prior to the demolition of WRH. Due to the weight of the plant required to install the anchors, existing floor slabs were propped from beneath to enable rigs to operate within the upper floor. This then hindered access to the ground floor while props remained in place.

Anchor locations were influenced by existing masonry cross walls. Furthermore, drilling holes for anchors would inevitably sever a portion of the existing rebar, thereby reducing the sectional capacity. In order to quantify this impact, the reduced capacity was calculated by assuming that one bar in fourteen would be severed on a statistical basis, a reduction of 7.1%.

To avoid over-stressing of isolated sections of the wall, one in every five anchors were installed, tensioned and locked off at design working load.



Figure 8. Ground anchor support system for wall.

The ground anchor system including anchor type, inclination and fixed length was designated a contractor design element. The specialist anchor contractor proposed an inclination of 35° for both upper and lower rows. This was found to have only a minor impact on the required anchor lock-off loads.

6.2 Verification and testing

Rigorous anchor testing in accordance with guidance presented in BS8081 was specified. The proof load for the suitability and acceptance tests was 150% of the design working load T_w , with load testing undertaken on working anchors only after the grout had reached a minimum crushing strength of 30N/mm².

Four ground anchors were specified for suitability testing at appropriate locations within WRH, in accordance with BS8081. Ground anchors meeting the suitability test criteria and which were not damaged in the testing process were permitted to be incorporated in the works as permanent anchors.

All ground anchors that were used in the permanent works were subjected to an on-site acceptance test, the loading procedure and the minimum periods of observations were carried out in accordance with BS8081. A typical anchor acceptance test result is presented in Figure 9.



Figure 9. Load-displacement graph for anchor no. 52.

Completed anchor installations are shown in Figures 11 and 12 below, with the waling beam visible along the upper row.

6.3 Movement monitoring

The wall was monitored for movement by the contractor before, during and following anchor installation with a system of 'tell-tale' gauges installed across movement joints at either side of WRH. These were monitored daily during critical construction phases and indicated that no movement beyond the predicted values was recorded. An example of such a gauge is presented in Figure 10.



Figure 10. Telltale gauge.



Figure 11. Lower level ground anchors.



Figure 12. Upper level ground anchors with waling beam.

7. CONCLUSIONS AND SUMMARY OF ADOPTED DESIGN APPROACH

This case study presents observations and conclusions of the structural remodeling of an existing retaining wall of limited provenance and specifically the design approach adopted to separate WRH from this wall supporting Manchester Road at its crest; WRH being demolished to make way for new development and was achieved with minimal disruption to the road.

Similar situations arise in city and town center development sites where existing buildings are required to make way for new buildings and adjacent highways or buildings are to be maintained live throughout the construction period.

Whilst details may vary for each project, this paper puts forward a process of appraisal and design development for consideration where scarce information is available regarding an existing structure, its construction techniques and the materials available at the time of its construction. This approach was found to be effective in this case. The key elements of the process were:

- (i) Thorough and early appraisal of existing information, however sparse, identifying any missing critical information.
- (ii) Undertake an intrusive site investigation to supplement the minimal or non-existent information on the existing structures, including section sizes, concrete compressive strength, reinforcement size and distribution, and geotechnical properties of the ground.
- (iii) Develop and maintain a positive, robust and cooperative relationship with other members of the design team, particularly the structural engineer.
- (iv) Undertake a back analysis based on the structural and geotechnical properties obtained to establish the most likely behavior of the wall. The choice of lateral pressure conditions behind the wall, namely controlled by active (K_a) or at-rest (K_o) conditions, is critical.
- (v) Embark on an engineering design process taking into account the modified structure and loads applied to it, and the range of ground parameters derived from the limited intrusive ground investigation in order to achieve an economic design which is fit for purpose and does not distress any element of the modified structure.
- (vi) Refine the scheme concept in the light of buildability, construction phasing and required aesthetics taking into account (iv) above, and develop the detailed design.

8. REFERENCES

- Arup. 2006. "Geo-Environmental Desktop Study, Waterfront Quarter, Huddersfield", ref 119046.
- Arup. 2009. "Intrusive Investigation at West Riding House and Assessment of Retaining Wall", ref 119046.
- Atkinson, J.H. 2007. *The mechanics of soils and foundations*, 2nd ed., Taylor and Francis, Abingdon, Oxon, UK.
- BS5400-4. 1990. Steel, concrete and composite bridges-Part 4: Code of practice for design of concrete bridges, British Standards Institution, London, UK.
- BS8002. 1994. Code of practice for earth retaining structures, British Standards Institution, London, UK.
- BS8081. 1989. *Code of practice for ground anchorages*, British Standards Institution, London, UK.
- CIRIA C143. 1995. The standard penetration test (SPT): Methods and use, CIRIA, 6 Storey's Gate, London.
- Soil Mechanics. 2007. Site Investigation Report No A7046.
- Soil Mechanics. 2010. Factual Report on Ground Investigation, Report No A9138.
- VHE Construction. 2009. Site Investigation Report No 515/03
- WML Consulting. 2009. Briefing Document to Coffey Geotechnics