New Control Building Envelope for British Gas.

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Discussion by

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INTRODUCTION

The authors have produced an interesting and informative description of the works necessary to provide blast protection to the completed control building for the British Gas North Morecambe Gas Terminal. Both the existing Control Building and the new blast resistant envelope required pile foundations. Some details were included and mention was made of associated vibro ground improvement. However, although fundamental to the performance of the structure under both blast and seismic conditions, details of the foundations were sketchy and many questions remained unanswered. This discussion is intended to present some brief details about ground conditions at the site, to describe the foundations in more detail and to provide some further background to the foundation design.

EXISTING PROCESS PLANT

Keller Ground Engineering successfully bid for advance civil and foundation works for the process plant at the British Gas North Morecambe Terminal site in Barrow. The contract started in September 1991 and was complete by October 1992, and is described in full by **Slocombe et al**[1]. Construction of the process plant items and associated structures began in 1992 and were substantially complete by early 1994.

Background information about the overall project including the offshore platform and pipeline is given by **Juren**[2] and **Spicer**[3]. A general description of the foundation works is given by **Ground Engineering**[4].

THE SITE AND GEOLOGY

Before development, the majority of the North Morecambe Terminal site comprised former settlement lagoons containing saturated pulverised fuel ash (PFA). Site investigation showed the PFA to be underlain by loose silty gravelly alluvial sand and soft clay, over more dense glacial sands, with glacial till and sandstone at depth. Typical ground conditions given by one of the pre-treatment cone penetration tests at the Control building site are shown in **Figure 1**.

SEISMIC DESIGN

As part of the early project feasibility studies, British Gas carried out seismic risk assessment which resulted in two levels of earthquake being specified for the design. General operating requirements were for a foundation solution able to withstand a 1 in

500 year design earthquake. Critical plant items and shutdown structures, including the Control Building, were required to survive a higher level 1 in 10,000 year seismic event. The assessment also identified the very real risk of liquefaction in both the PFA and the underlying loose alluvial and glacial sands. Foundations were required to cater for the dynamic earthquake loadings and soil liquefaction. The foundation scheme put forward for the critical plant items and shutdown structures was based on the use of vibro densification techniques using stone columns with short cast in place piles driven into the treated soils, as illustrated schematically in **Figure 2**. Vibro techniques were required to densify the alluvial and glacial sands to prevent liquefaction during the design earthquake.

Background to the seismic design for the foundations is given by **Raison et al**[5].

PILE DESIGN

In April 1994, Keller were approached to provide advice on foundations for the new blast envelope, and were subsequently appointed as sub-contractor for additional vibro and piling works.

Because of the proximity of the partially commissioned Control Building, driven cast in place piles as used elsewhere on the site, could not be used for the new blast resistant envelope. Foundations therefore comprised continuous flight auger piles installed into soils previously treated with vibro stone columns. Stone column and foundation pile layout for the Control Building and new blast envelope is given in **Figure 3**. Vibro treatment was carried out using up to 1m diameter stone columns at 2m centres to 18.5m depth. Treatment was confirmed using pre and post-treatment static cone penetration tests.

Because of the seismic design requirements, 600mm diameter CFA piles were proposed reinforced with Universal Bearing Pile sections, **Figure 4**. UBP sections were necessary to cater for the high shears and bending moments under seismic loading conditions, with the ability to maintain sufficient ductility to prevent loss of axial load capacity or collapse during PFA liquefaction. The use of UBP sections was particularly advantageous for dealing with blast loading conditions where horizontal and tension loadings were more onerous.

For normal operating conditions piles were installed for up to 1000kN axial working load. Blast conditions were expected to induce tension loads up to 400kN and horizontal loads typically up to 135kN per pile. Design lengths for the CFA piles were longer than those computed for the existing driven piles. Design assumed a clay toe and a minimum factor of safety of about 2, but were expected to achieve a factor of safety greater than 3 if founded in sand. Piles were founded 19m below ground level and reinforced with 15m long UBP sections.

PILE TESTING

As part of the works, a non working prototype CFA pile was installed and tested to confirm axial compression, tension and lateral load capacity. The test pile was founded in sand at a depth of 14.5m. Instrumentation including embedded concrete strain gauges, rod extensometers and an inclinometer access tube was installed with the UBP section. Testing was carried out using computer logging and control following specified loading and unloading cycles with a final constant rate of penetration cycle for the axial compression test.

Figure 5 shows the measured pile load versus head settlement relationship for the axial compression test and the tension test. Both tests confirmed load capacities in excess of those computed from the site investigation data. Back analysis confirmed that even when bored with a clay toe, working piles would develop sufficient shaft friction capacity for both seismic and blast loading conditions.

Figure 6 presents the horizontal load deflection relationship recorded during the lateral load test. **Figure 7** gives the corresponding displacement profiles measured using an inclinometer at each peak load stage during the test. Measurements show fixity of the pile at about 4m depth. Back analysis of the results showed a stiffer behaviour than anticipated and suggested a greater horizontal load capacity.

CONSTRUCTION

As described in the paper, design and construction of the foundations was carried out concurrently with the structural design of the new blast envelope. Despite the difficult programming demands, foundation works were completed on time and within budget and are believed to have assisted in the successful completion of the structural works by the other members of the team.

REFERENCES

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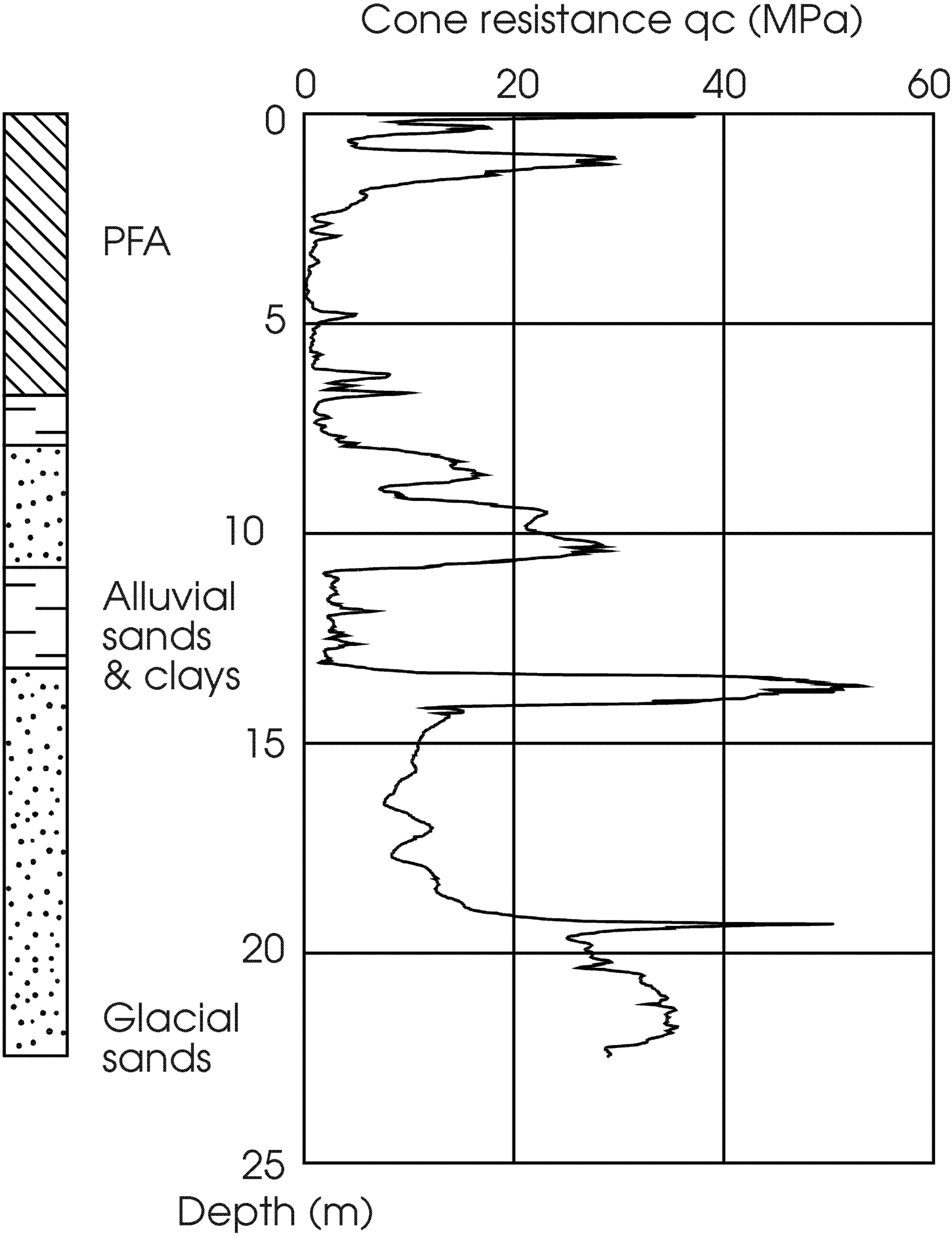

Figure 1. Typical ground conditions

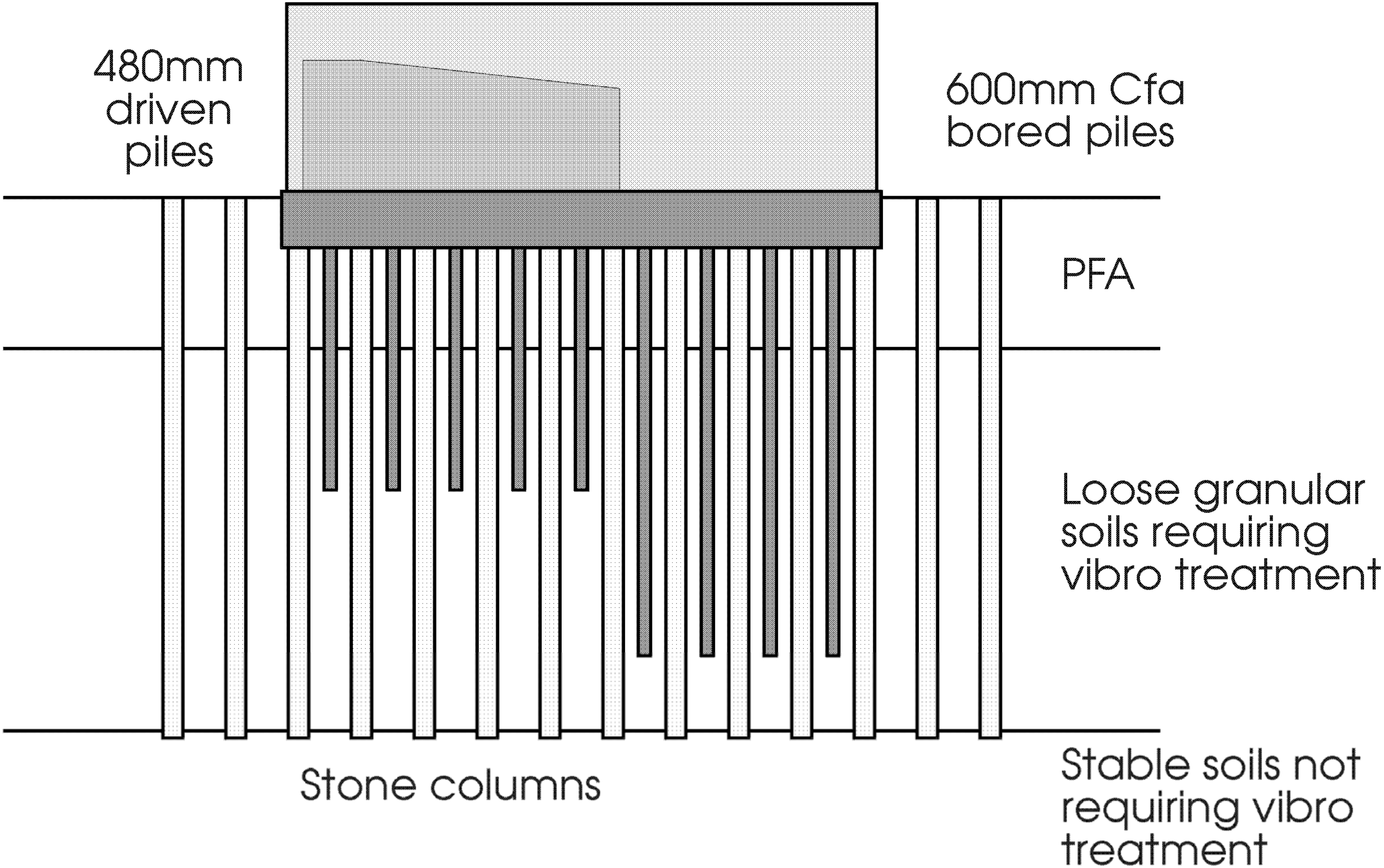

Figure 2. Proposed foundation solution

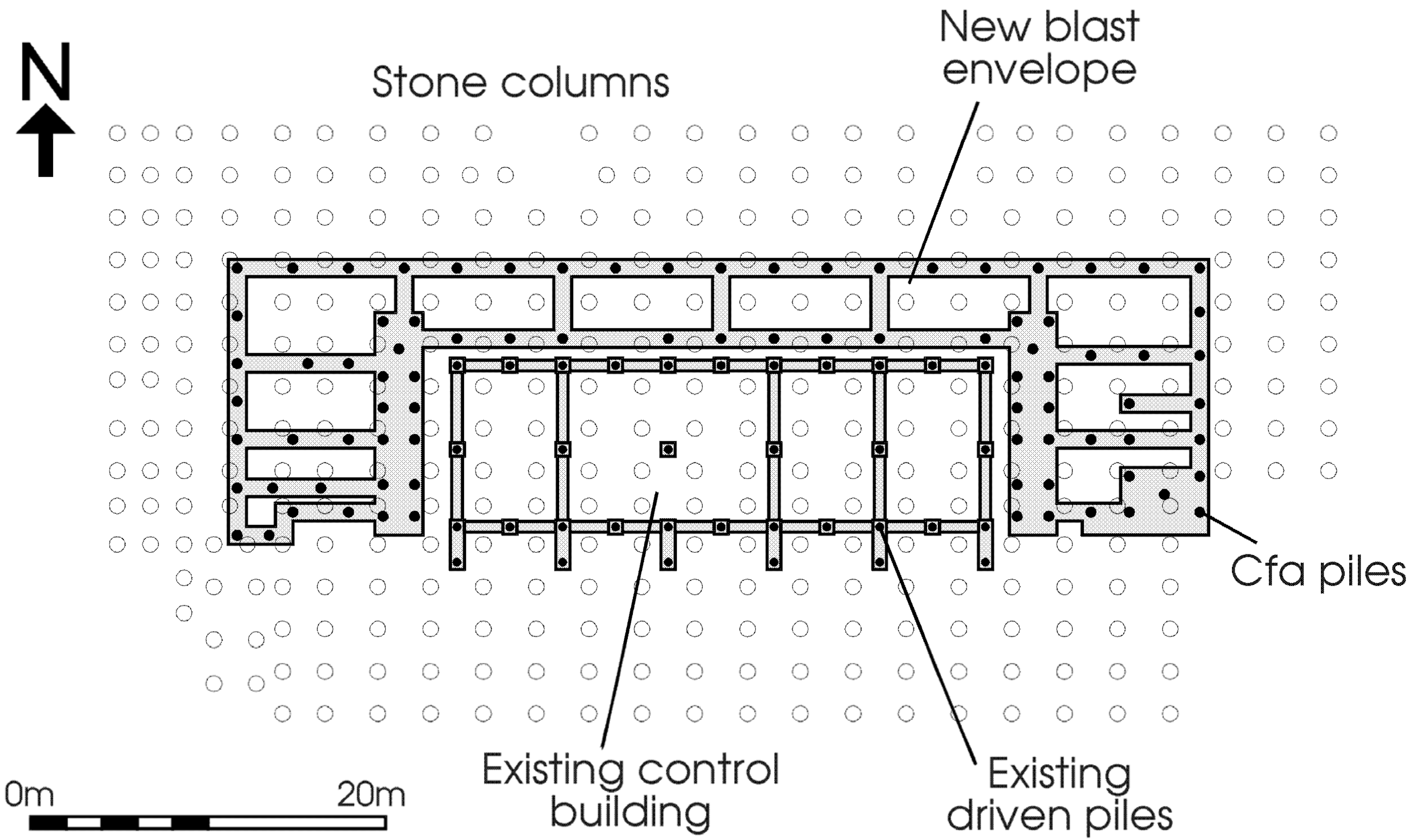

Figure 3. Stone column and pile layout

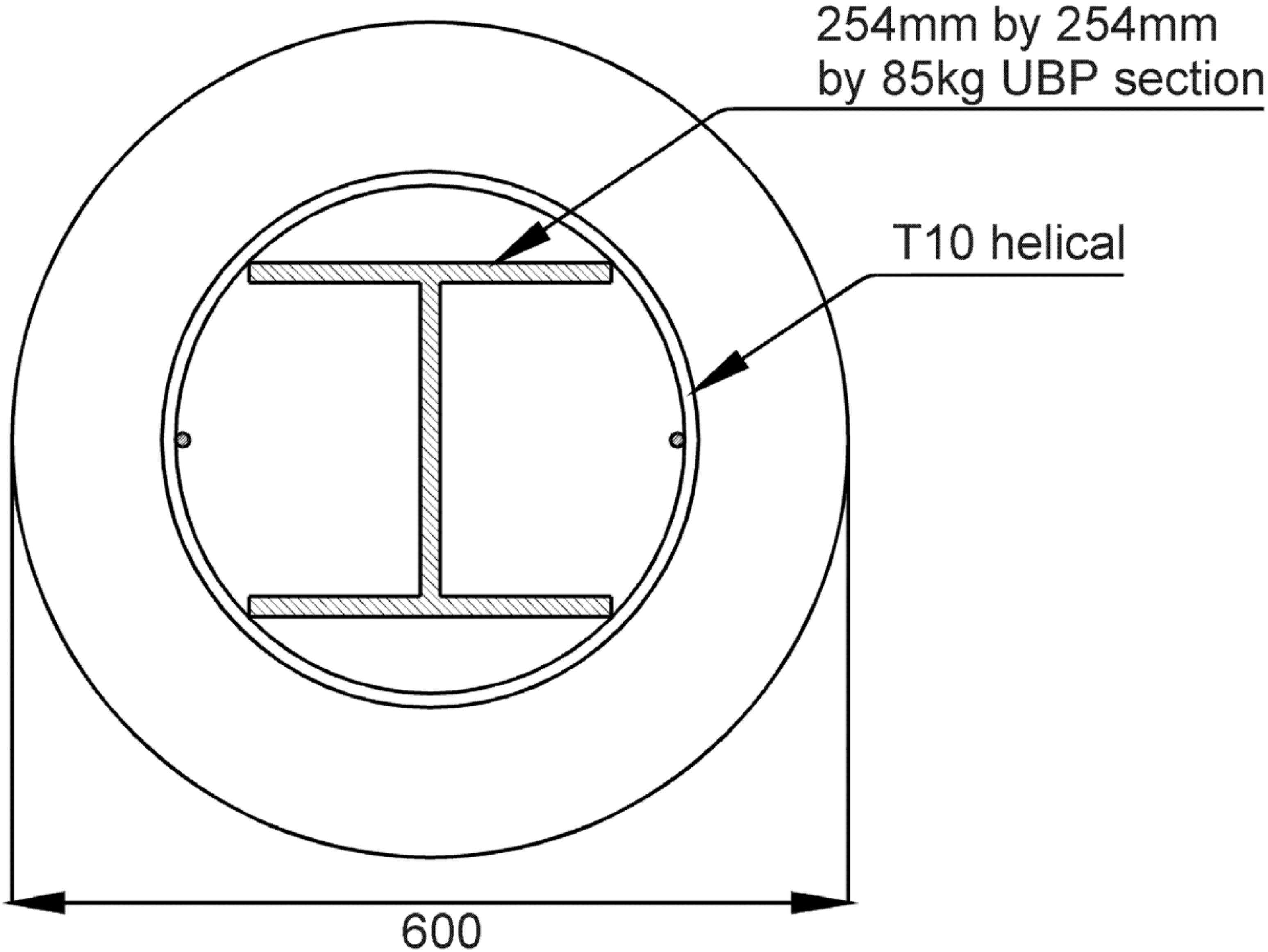

Figure 4. Proposed pile details

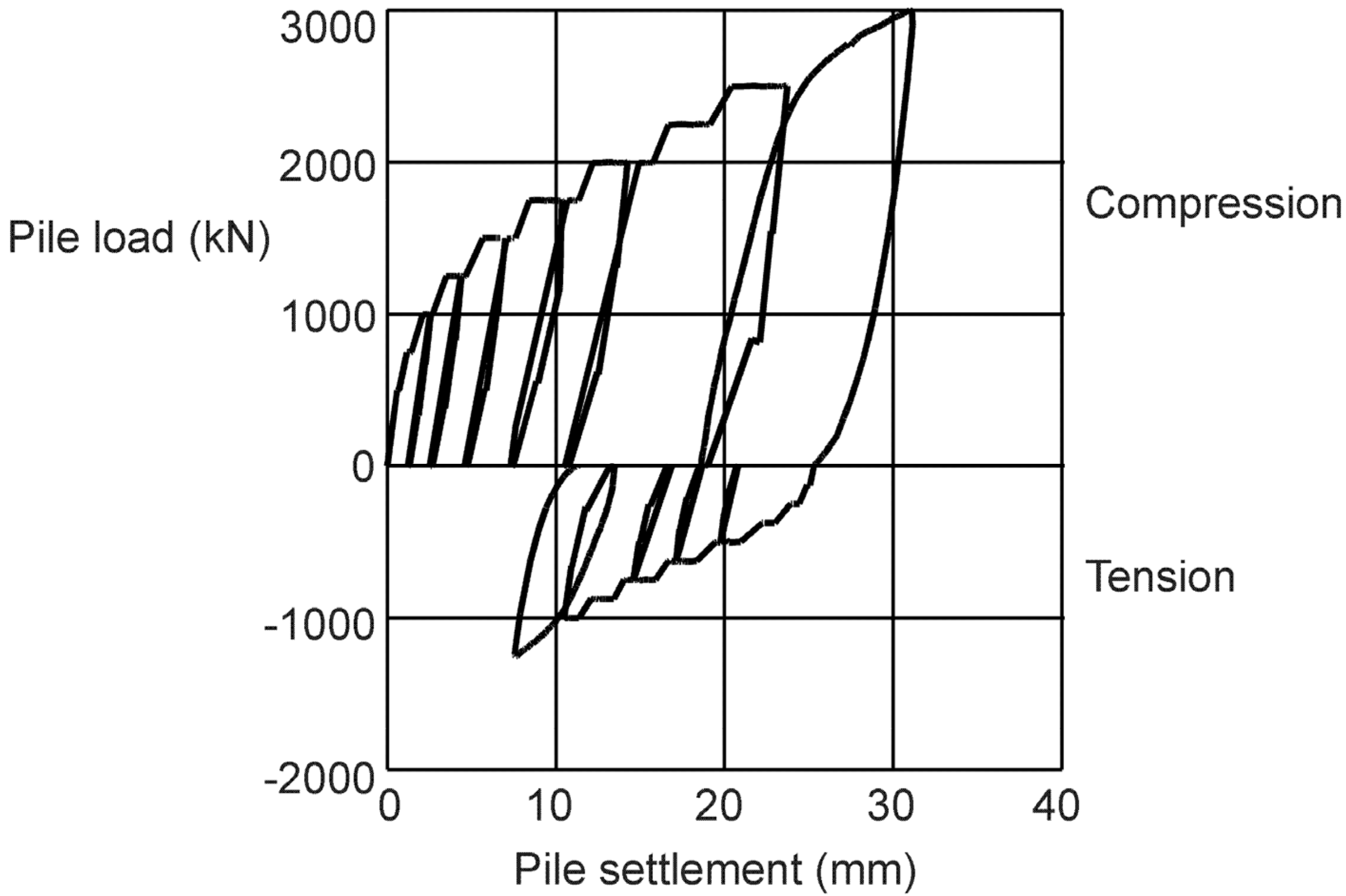

Figure 5. Load settlement relationship

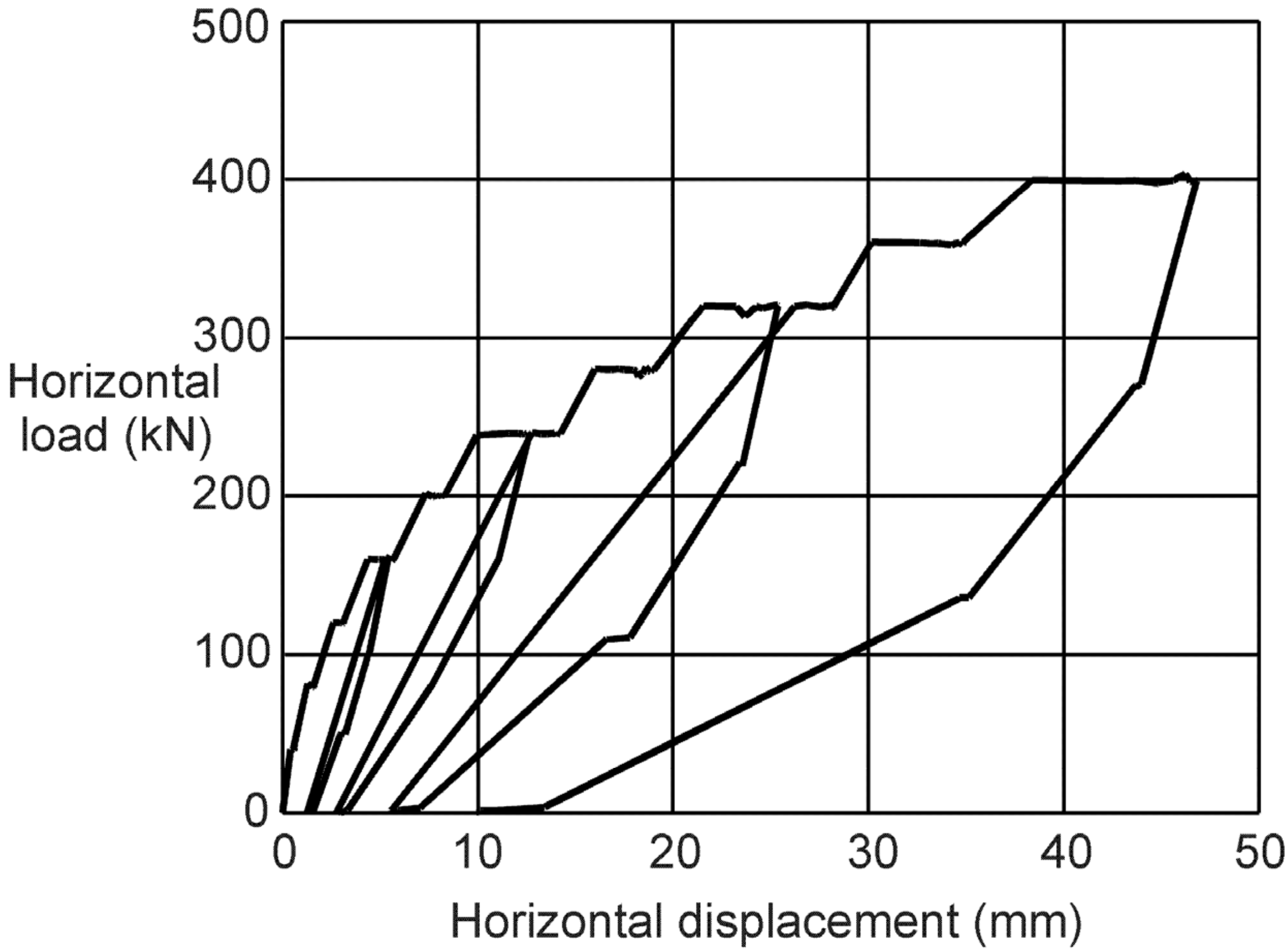

Figure 6. Horizontal load displacement relationship

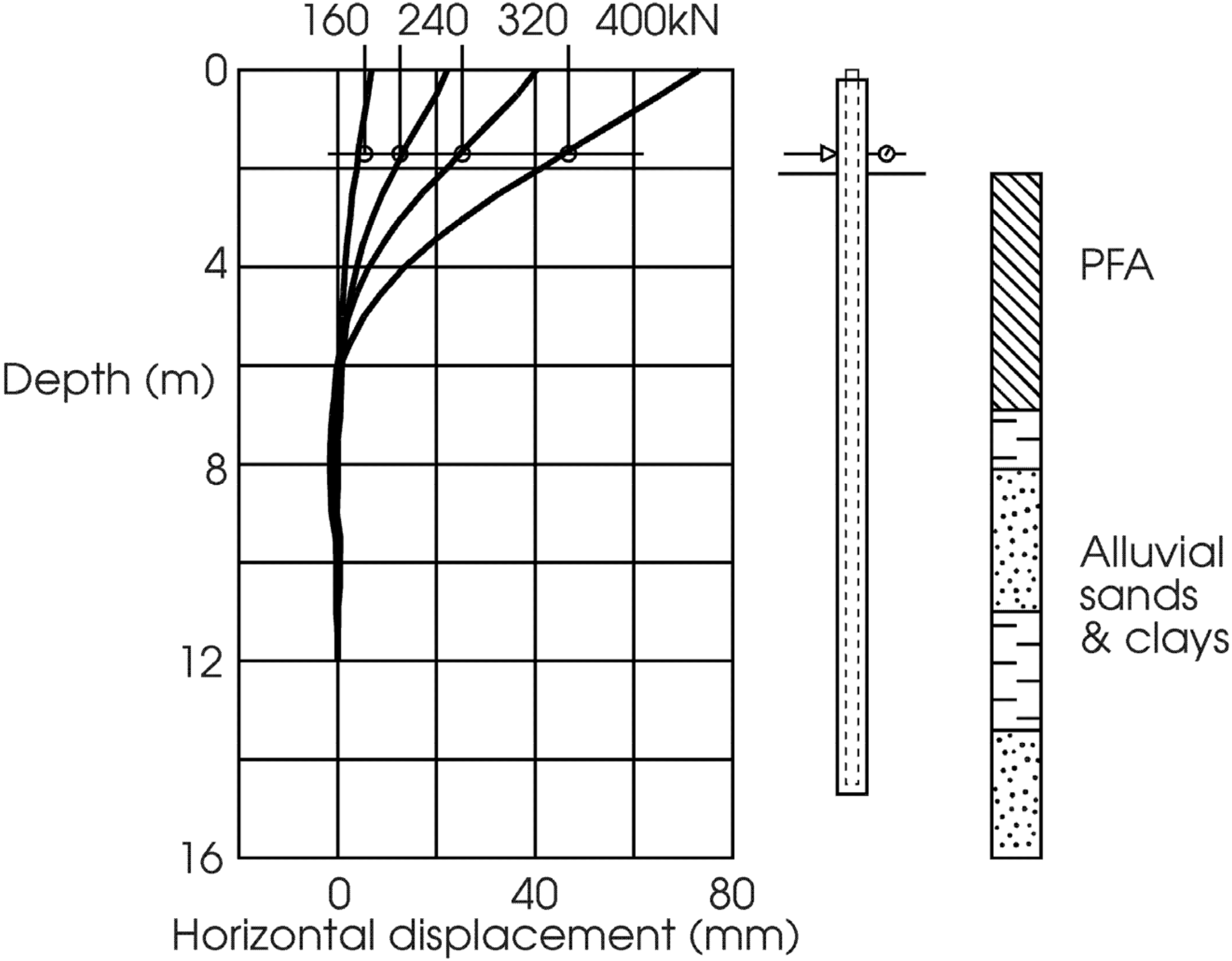

Figure 7. Inclinometer measurements