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Chapter 98

Pile capacity testing

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As there is still uncertainty in accurately predicting the performance of piled foundations based upon design calculations there remains a need to test piles. Several methods of pile testing are available some of which have been in common usage for many years, while others are relatively recent developments. Static pile testing is a well understood and simple test technique that has been in use for many years. This approach has the benefit of directly producing test results but is hindered by the increasing size of associated testing infrastructure as pile capacity increases. A recent variation of classic top-down static testing is bi-directional testing which relies on the incorporation of a specialised loading jack or jacks in the pile shaft at some depth below ground surface. This technique has the ability to apply test loads that greatly exceed those possible in other pile test types by effectively using one portion of the pile capacity to test against the other or others. Alternative pile testing techniques come in the form of rapid and dynamic load tests that have the benefit of quick testing and reduced testing infrastructure but require more complicated analysis and interpretation techniques due to the rapid/dynamic nature of the tests.

98.1 An introduction to pile testing

Although pile design has advanced in recent decades, the determination of axial pile capacity is still dependent on the use of empirical correlations. As a result, it is only possible to estimate capacity to ±30% in many soil types (Randolph, 2003) whereas in rock, capacity may be significantly under predicted. Owing to the uncertainty associated with predicting load-settlement behaviour using existing design methods, it is common practice to carry out pile load tests for verification.

The information obtained from pile load testing may be used in a number of ways (Poulos, 2000) including:

- construction and quality verification;
- verification of design information;
- to allow for a more refined or confident design with potential cost savings for subsequent piling works.

As pile testing may prove expensive (especially on small contracts), the need for pile testing may be considered in terms of risk reduction. On larger sites, where significant numbers of piles are being installed, pile testing may be considered from a perspective of potential cost saving. Through testing, improved design parameters may be determined resulting in optimisation of the piles through a reduction in length, for example. The pile testing strategy recommended by the Federation of Piling Specialists (FPS) (2006)in their Handbook on Pile Load Testing is shown in **Table 98.1**. Similar pile testing recommendations can be found in BS8004:2015 for preliminary and working piles in low and medium risk situations.

Eurocode 7 allows pile design based upon a pile load test(s) and states that pile tests should be undertaken in the high- to medium-risk situations highlighted in **Table 98.1**. Eurocode 7 also allows the characteristic resistance of axially loaded piles to be derived directly from pile load tests with the level of correlation increasing with increasing number of piles tests (from one to five). For five pile tests or more direct correlation is allowed with the measured test results which in turn are reduced by a partial factor to obtain the design resistance (see the relevant sections in BSEN 1997 (2004) otherwise known as EC7, and the relevant National Annex for further detail). This prescriptive approach to factoring may be considered an improvement over the *ad hoc* manner in which pile load tests have previously been used to influence safety factors. Further information, with respect to pile testing as part of the Eurocode framework, is available in BS EN1536:2010+A1:2015, *Special Geotechnical Works: Bored Piles*. Piles may be tested at different stages throughout a project depending on how the outcome of the test will be used. For instance, preliminary or trial piles are tested prior to the main works or pre-contract and are designed to validate design and achievable performance along with checking construction techniques in certain ground types. It is important that preliminary piles are tested sufficiently in advance of the main works such that findings from the tests can be incorporated in working pile designs. Piles tested that will form part of the final structure are referred to as working piles.

As well as the stage of the project at which piles should be tested, it is also important to consider the time after installation at which piles should be tested. For instance, driven or displacement piles installed in clay cause significant disturbance of the ground. This results in changes to the local effective stress regime which with time may recover depending on the soil's permeability and the size of the pile (Jardine *et al*., 2005). For instance, the *ICE Specification for Piles and Embedded Retaining Walls* (2017) (often referred to as SPERW) suggests a minimum of four days between installation and testing for cast insitu piles to avoid damage and achieve suitable cube strength, although it does go on to say that a 12 hour delay between installation and testing is adequate for driven piles which appears to ignore the potential for setup (gain in strength with time) or set down (reduction in strength with time) in different soil types caused by the installation process. In contrast, Fleming *et al*. (2009) suggest 1 to 3 weeks between installation and testing depending on soil type and experience where up to 28 days may be adopted in practice. BSEN ISO 22477-1 recommends 7 days between installation and testing for "coarse" soils and 21-28 days in "fine" soils, dependent on if the pile is bored or displacement type.

Pile load testing methods include static tests, dynamic tests and kinematic or rapid load tests. There are also alternative static load test methods such as bi-directional testing in the form of the Osterberg Cell (O-Cell). Generally, static pile testing methods are expensive and time consuming (Fleming *et al*., 2009) becoming more so with increased load requirements, but have the advantage of simple analysis and interpretation. Conversely, dynamic and rapid load testing methods are quick to carry out and become cheaper than static tests with increased load requirements but require more specialised equipment and analysis. Comparisons of some of the typical pile test characteristics are shown in **Table 98.2**. Note that this table is based upon current typical approaches to testing and does not dictate use of the techniques; for instance, dynamic tests may be undertaken on cast *in situ* piles but this may not be a frequent occurrence in practice or certain parts of the world.

Characteristics of the piling works	Risk level	Pile testing strategy
Complex or unknown ground conditions No previous pile test data New piling technique or very limited relevant experience	High	Both preliminary and working pile tests essential 1 preliminary pile test per 250 piles 1 working pile test per 100 piles
Consistent ground conditions No previous pile test data Limited experience of piling in similar ground	Medium	Pile tests essential Either preliminary and/or working pile tests can be used 1 preliminary pile test per 500 piles 1 working pile test per 100 piles
Previous pile test data available Extensive experience of piling in similar ground	Low	Pile tests not essential If using pile tests either preliminary and/or working pile tests can be used 1 preliminary pile test per 500 piles 1 working pile test per 100 piles

Table 98.1 Typical pile testing strategy based upon risk levels. Reproduced from the Federation of Piling Specialists (FPS) (2006). Handbook on Pile Load Testing.

Type of pile test			
Characteristic	Static[1]	Rapid	Dynamic
Load duration	1–24 hours	100 milliseconds	7 milliseconds
Tests per day	1	2–6	8
Reaction mass required (as a percentage of pile capacity)	120%	5–10%	1.5–2%
Time needed for results	Directly	10 minutes	4 hours
Pile types tested			
precast	yes	yes	yes
cast *in situ*	yes	yes	yes
tubular steel	yes	yes	yes
Perceived reliability	high	experience too limited to assess	intermediate
Cost[2] (Pounds per kN of pile capacity)	0.42–0.75	0.45	0.05–0.1
Cost per pile	850–1500	400	50–200

(1) Static definition here does not include O-Cell (top-down).
(2) Based upon testing at 1000–2000 kN capacity piles on the same site, note dynamic test costs assume a pile driving rig is present on site. Costs given in GB pounds.

Table 98.2 Comparison of typical features of pile testing techniques. Data taken from Hoelscher and van Toll (2009)

98.2 Static pile testing

98.2.1 What is static pile testing and what types of test are there?

The most common method of pile load testing, static testing, may take one of two forms. These are the maintained load test (MLT) and the constant rate of penetration test (CRP). The names of the two tests are derived from their methodology. Since the invention of tests such as the O-Cell method, these types of testing are often referred to as top-down as the loading is applied at the head of the pile.

98.2.2 Maintained load compression test (ML or MLT)

This type of test is often referred to using the acronyms ML or MLT. ML testing works by applying and maintaining increments of load to the head of the pile for a minimum specified time (**Table 98.3**) and until a specified rate of settlement criterion is satisfied (**Table 98.4**), at which point the load is either increased or reduced (Tomlinson and Woodward, 2008). According to SPERW (2017) the minimum time for holding of an increment typically varies from 30 minutes to 6 hours, with unloading increments typically held for 10 minutes. The approach in SPERW (2017) varies from that in BS EN ISO 22477-1:2018 but it is noted that minimum hold periods are recommended rather than prescribed. An example of the pile response seen during an ML test is shown in **Figure 98.1**.

Generally, this test method is not always used to prove the pile ultimate load capacity or generate ‘plunge’ as it is difficult to maintain constant load at high settlement rates. Additionally, it is typical to increase the pile load in 25% increments (**Table 98.3**) of the working design verification load (DVL, see definition in **Table 98.3** and specification section later), which may mean the application of many hundreds of kN between load increments. It is then possible for the actual ultimate load to be missed resulting in an underestimation of ultimate capacity as the pile plunges at a higher load increment. To avoid the underestimation, the load increment may be reduced to 10% DVL throughout the test or when load stability issues occur as is recommended for testing of preliminary piles (SPERW, 2017). SPERW (2017) also includes additional guidance on shortened duration MLT proof testing for steel screw piles which differs from other pile types.

Load*	Minimum time of holding load for a single-cycle pile test	Minimum time of holding for a multi-cyclic pile test
25% DVL	30 minutes	30 minutes
50% DVL	30 minutes	30 minutes
75% DVL	30 minutes	30 minutes
100% DVL	6 hours	6 hours
75% DVL	n/a	10 minutes
50% DVL	n/a	10 minutes
25% DVL	n/a	10 minutes
0% DVL	n/a	1 hour
100% DVL	n/a	1 hour
100% DVL + 25% F_{rep}	1 hour	1 hour
100% DVL + 50% F_{rep}	6 hours	6 hours
100% DVL + 25% F_{rep}	10 minutes	10 minutes
100% DVL	10 minutes	10 minutes
75% DVL	10 minutes	10 minutes
50% DVL	10 minutes	10 minutes
25% DVL	10 minutes	10 minutes
0% DVL	1 hour	1 hour

*F_{rep} denotes the representative action, previously referred to as SWL, Specified Working Load; DVL denotes Design Verification Load

Table 98.3 Minimum loading times for a maintained load compression proof test. Reproduced from Institution of Civil Engineers (2017) (SPERW, 2017)

Pile head displacement range	rate of settlement criteria
< 10 mm	≤0.1 mm/hour
> 10 mm	≤0.2 mm/hour

Table 98.4 Rate of settlement to be used along with minimum hold times

Figure 98.1 Example of CRP and MLT static load tests on a 12 m long, 600 mm diameter cast in situ bored pile in glacial till (see Brown *et al.*, 2006 for more information on the pile installation and testing). Data taken from Brown *et al.* (2006)

98.2.3 Constant rate of penetration testing (CRP)

The CRP test varies from the ML test in that a varying load is applied to the pile to maintain a constant rate of penetration. The rate of penetration is typically chosen to reflect the main soil type that the pile installation encounters (**Table 98.5**). Due to these penetration rates, tests are completed relatively quickly. For instance, a 600 mm diameter pile installed in clay pile can be taken to a penetration equal to 15% of the pile diameter (90 mm) in 2.5 hours. It can be seen in **Table 98.5** that the rates used for CRP in US practice may be 50% slower or faster than those specified for UK use. It should also be noted when comparing tests or correlations from the US that there is also a different ML test referred to as the quick load test method (QLT) where load increments are only held for 2.5 minutes (ASTM D1143/ D1143M-20).

The CRP test is not considered appropriate for most testing situations and is typically reserved for research purposes and has more recently been removed from revisions of codification (as has guidance on pile penetration rates adopted during a CRP test). Although testing is faster, it may require greater capacity from the loading and reaction systems to produce plunge. There are also reservations about the relatively high penetration rates and short test duration (rate effects) especially where piles are installed in clay. It has been shown that as the penetration rates in CRP increase so does the ultimate pile capacity and stiffness (**Figure 98.1**). In terms of representing meaningful structural loading rates, it may be argued that the loading rate should be selected based upon the permeability of the soil specific to the pile installation and effective drainage path length (often related to pile diameter). It should also reflect whether the desired behaviour to be proven is drained or undrained (Brown and Hyde, 2008).

Major soil type	Rate of penetration mm/s (mm/min)		
	SPERW 2017	BS8004:1986	ASTM D1143
Fine-grained soils (e.g. clay)	0.01 (0.6)	0.0125 (0.75)	0.0042–0.021 (0.25–1.25)
Coarse-grained soils (e.g. sand or gravel)	0.02 (1.2)	0.025 (1.5)	0.0125–0.042 (0.75–2.5)

Table 98.5 Examples of different pile penetration rates specified for CRP testing. Note BS8004:1986 has now been superseded/withdrawn, information not included in latest revision.

98.2.4 How it works and the various test set-ups

Static pile testing systems typically require a structure to react against to allow the application of load to the pile. A typical reaction or anchor pile type arrangement for pile testing is shown in **Figure 98.2**. Alternatively, the reaction to the hydraulic jack can be provided by placing kentledge (reaction mass) above the jacking arrangement (**Figure 98.3**). If the pile is a preliminary pile, i.e. one that is required to validate construction performance prior to construction of the working piles, then either arrangement of pile test would be suitable. Where load tests are required on piles that will form part of the final structure (working piles) then the test arrangement using kentledge would appear more appropriate but often reaction piles are used and greater care is required with respect to the spacing between the reaction piles and those under test. Unfortunately, if several working pile tests are needed, multiple individual test arrangements may be required to avoid time delays. Greater detail regarding pile testing procedure and equipment arrangements is given by Weltman (1980) and FPS (2006).

Figure 98.2 Top-down static pile testing arrangement utilising a tension pile reaction system. Modified with permission from CIRIA PG7 Weltman (1980), www.ciria.org

Figure 98.3 Top-down static pile testing arrangement utilising water-filled tanks as a reaction system. The testing arrangement is completely obscured by the tanks. Photograph courtesy of Deltares; all rights reserved

Application of the load to the test pile is usually undertaken using a large hydraulic jack with manual or automated control of hydraulic oil flow to allow either a maintained load to be applied or varying load for constant penetration rates (**Figure 98.4**). Historically load has been measured with a manually read calibrated load column placed between the jack and the reaction system (**Figure 98.4**). It is now more likely that an automated strain gauge-based load cell will be used which produces readings that can automatically be logged by a PC or datalogger. Similarly, the piles' deflection has been measured using manually read dial gauges (mounted on a reference beam placed three diameters or 2 m from the test pile) whereas these have been typically replaced by displacement transducers that can be logged by a PC or datalogger (Fleming *et al*., 2009). Measurement by displacement transducers or dial gauges should also be verified by optical levelling to an accuracy of 0.2 mm. Developments in monitoring equipment along with reliable and cost-effective servo hydraulic control systems (which allow true stress and displacement controlled tests to be undertaken) have led to the development of fully automated test rigs that can be operated remotely (England, 2002). Automating such a system means that staff spend less time on an individual site and less time travelling between sites.

Another advantage of automation is that the safety of site operatives is enhanced as they do not have to spend long periods close to highly stressed structures which in extreme cases have failed causing injury. There is also less need for shift working which may result in operatives working on deserted sites during the hours of darkness.

Figure 98.4 Pile head loading arrangement showing loading jack placed on the pile with calibrated load column above with spacers to the reaction frame. Note the dial gauges for settlement measurement in contact with the head of the pile

98.2.5 Static tension tests

Tension or uplift testing of piles (**Figure 98.5**) utilises similar equipment and testing procedures to compression testing although a single hollow jack or two jacks may be used to avoid inducing bending stresses in the pile. Dependent upon beam arrangement and surface soil strength in compression, resisting piles may be required. This will not be the case where the surface is stiff enough to accommodate the compressive force imparted, in which case trestles and spreader plates may be sufficient. Connections to cast *in situ* piles are similar to those employed in tensile reaction piles whereas preformed steel piles may require additional welded brackets (Tomlinson and Woodward, 2008; Fleming *et al*., 2009). It is normal to test more than one pile that may be subject to tension and at least 2% of piles where a large number are subject to tension.

Figure 98.5 Tension pile test. Note loading jack above the reaction beam. Photograph courtesy of Cementation Foundations Skanska Ltd; all rights reserved

98.2.6 Static lateral load testing

As well as tensile load, piles are often subject to combined lateral, vertical and moment loading which in certain circumstances, such as foundations for offshore structures, may be cyclic in nature. It is not easy to achieve such complicated load paths in full-scale pile tests but both monotonic and cyclic lateral tests may be specified (Burd et al., 2020). Typically, an adjacent pile(s) is used as reaction with a jack placed in between one or more reaction pile(s) and the test pile (**Figure 98.6**). Where cyclic loading is required automated jacks and monitoring systems are recommended. Similar load/deflection measuring techniques may be used as per axial pile load tests, but it is also advisable to measure relative movement of the test pile to the reaction piles and pile head rotations. In addition, valuable information for test analysis can be obtained by instrumenting the pile (Reese and van Impe, 2011) to allow determination of moments with depth. When selecting the number of lateral piles to be tested, close attention should be given to the variability of the ground over the top few metres of the test pile as this zone has significant effect on the lateral pile behaviour.

Figure 98.6 Lateral load pile tests showing piles being jacked off each other. Displacement transducers (mounted on the white reference frame) to monitor lateral deflections can be seen against the test pile on the right-hand side of the image. Photograph courtesy of Cementation Foundations Skanska Ltd;

98.2.7 Advantages and disadvantages of static testing

The advantages of static testing lie in the simplicity of the test and it having been a long accepted technique. Generally, interpretation is simple and results can be produced quickly without the need for specialist interpretation or in some cases with only limited knowledge of the ground conditions.

Static tests are relatively slow. For instance, a typical ML test generally takes a minimum of 19 hours but may take much longer depending on the particular test specification and the settlement rate characteristics of the pile under test. This also neglects the time required for setting up the test equipment. This has programme issues and implications regarding site safety with the need in some cases for 24-hour working although system automation can further reduce risks.

The main disadvantages of static testing stem from the need to have enough reaction at ground level to apply loading. This results in infrastructure-intensive reaction frames connected to tension piles or kentledge (**Figures 98.2** and **98.3**). Such systems are time consuming to construct, expensive and may have significant space requirements both in terms of footprint and materials handling. The need for such large reaction (120% of anticipated ultimate capacity, **Table 98.2**) also means that the reaction system is highly stressed during testing, resulting in potential safety concerns. This may result in the testing of reduced diameter preliminary piles or “model” piles compared to those proposed as contract piles to further understand the components of pile resistance and test verify design approaches. More recently for very large piles, it has been proposed to undertake 'multi suite' testing where a series of preliminary piles that have different elements of the pile capacity that are isolated or significantly reduced in certain strata through the use of sleeving (Hardy *et al.*, 2019), low friction bitumen coatings or including compressible bases. In this way the pile diameter may be kept as designed and the limiting shaft friction for each strata and base capacity established from the different tests. The multi-suite approach can then allow valuable information to be gained on pile performance in different layers whilst adopting a smaller reaction frame and testing system.

One of the possible drawbacks of the anchor pile or kentledge-based reaction systems is that they may interfere with the behaviour of the pile under test. Poulos (2000) suggests that the use of kentledge may cause an increase in test pile capacity and stiffness. Predictions in sand suggest that pile capacity and stiffness might be increased by 10–20% due to the presence of kentledge (Poulos and Davis, 1980). With this in mind, SPERW suggests that there should be a minimum separation of 2 m between the kentledge supporting arrangement or three times the test pile diameter (BS EN ISO 22477-1:2018 suggests the maximum of 2.5D or 2.5 m). For a reaction pile arrangement where the reaction piles are shorter than the test pile, there should be a minimum separation of 2–3 m (depending on guidance consulted) or 2.5 to three times the test pile diameter. If the reaction piles are longer than the test pile (or inclined) the spacing should be increased to five times the diameter of the largest pile (test or reaction) unless the base capacity of the test pile is 20% less than that of the reaction piles. In the case of tension tests, any ground beams or adjacent compression piles used to support the reaction system should be at least three times the test pile diameter away from the test pile.

98.2.8 Interpretation of static tests

Inspection of the load-settlement curves determined from pile testing can be used in several ways. Firstly, by inspection the shapes of the curves may hint at the adequacy or integrity of the installation and highlight problems occurring during pile formation. Additionally, the result may be used to check performance criteria.

98.2.8.1 Settlement criteria

Pile design is typically based upon determining both ultimate and serviceability limit cases or states. When assessing performance from pile tests, it seems that settlement criteria at working loads may be given more attention (in an onshore setting) than ultimate pile capacity. This is probably a result of ultimate capacity being associated with settlements that would cause serviceability damage to structures.

For working load tests, historically SPERW suggested that settlement criteria should be in the range of 5–10 mm at the design verification load (DVL) and 15–25 mm DVL + 0.5 F_{rep} (F_{rep} is the representative action, previously referred to as the specified working load, SWL), and that this this should be assessed based on the specific building type. In the most recent version of SPERW these values seem to have been removed which maybe an attempt to reinforce the need for these values to be determined on a structure specific basis but there may still be a need to specify limits.

98.2.8.2 Determination of ultimate capacity and test termination

Another potential barrier to using ultimate axial pile capacity to define performance seems to lie in the various definitions of what constitutes the ultimate pile load or capacity. For example, Tomlinsons and Woodward (2008) list seven different recognised techniques. These may be based upon the load at a certain settlement defined by the pile diameter (for instance, 10–15% of the test pile diameter, BS8004:2015) or a feature related to the shape of the load-settlement curve such as the load at which settlement continues to increase without further increase in load. Definitions such as the latter may be problematic; for instance, in granular soils it is common to see gradual increases in capacity with increasing settlement, i.e. the criterion is never met.

EC7 seems relatively vague on this point but does state that where there is difficulty in determining the ultimate capacity a value equal to 10% of the diameter should be assumed. SPERW (2017) avoids the use of a specific settlement criterion and defines the ultimate capacity in MLT as the maximum load that can be applied whilst achieving the specific settlement rate criteria (**Table 98.4**). Conversely, though, CRP tests in SPERW (2017) are terminated on the basis where loads are constant or reducing for at least 10 mm or a settlement equal to 15% of diameter is achieved.

If a preliminary pile does not reach ultimate capacity, then results may need to be extrapolated to give an indication of the expected capacity. Methods proposed by Chin (1972) and Fleming (1992) referred to as hyperbolic methods are often used for this with varying degrees of performance for particular pile and soil types (BS8004:2015). The Chin method is based on the assumption that measured pile load-settlement behaviour can be represented by a single hyperbolic function such that the ultimate capacity (U) of a rigid pile can be represented by

$$U = \frac{\Delta}{(\Delta/P) - K} \qquad (98.1)$$

where Δ represents the pile head settlement at any pile head load (*P*), and *K* is the intercept on the horizontal axis (Fleming, 1992). Chin observed that if Δ/P was plotted against settlement (Δ) a straight line was obtained (**Figure 98.7**). The gradient of the straight line is the ultimate capacity of the pile (Equation 98.1). **Figure 98.7** also shows the CRP test previously presented in **Figure 98.1** with the ultimate pile capacity predicted from the Chin analysis shown as an asymptote at 2323 kN.

Figure 98.7 Chin analysis of a CRP static load tests on a 12 m long, 600 mm diameter cast in situ bored pile in glacial till

The form of the hyperbolic function was then improved upon by Fleming (1992) through incorporating parameters from back analysis of a large pile testing database and the effects of pile shortening by recognising that there are potentially two separate behaviours and consequently two hyperbolic functions. Fleming (1992) separates consideration of pile shaft (denoted subscript *s*) and base (denoted subscript *b*) components such that:

$$\Delta_s = \frac{M_s D_s P_s}{U_s - P_s} \qquad (98.2)$$

where the pile diameter (D_s) has been incorporated to recognise its influence on load settlement behaviour and M_s is a dimensionless flexibility factor with low variance with typical values between 0.001 and 0.0015 (Azizi, 2000). Pile base settlement is based upon the assumptions made for linear-elastic settlement of a footing and assessment of soil secant modulus (E_b) from a real load/settlement relationship under a footing at one quarter of the ultimate stress (U_b /4).

$$\Delta_b = \frac{0.6U_bP_b}{D_bE_b(U_b - P_b)} \tag{98.3}$$

Note that equations 98.2 and 98.3 above are for consideration of a rigid pile only. See Fleming (1992) for inclusion of elastic shortening. A useful summary of typical soil secant modulus values (E_b) can be found in Azizi (2000).

More detail of these techniques and interpretation of the results of static testing can be found in Fleming *et al*. (2009) and Tomlinson and Woodward (2008). BS8004:2015 also refers to other methods such as the Davisson's offset limit and the Butler and Hoy's slope and tangent method.

One of the aims of pile testing may be to verify or update design parameters and techniques. Where this requires specific knowledge of separate shaft and tip resistance, mobilized shaft frictional resistance with depth and settlement it is recommended that the test pile is appropriately instrumented rather than attempting to derive such information based upon pile head measurements alone.

98.2.9 Specifying a static pile test

The magnitudes of the applied load increments for an ML test (**Table 98.3**) are chosen to verify the ability of the pile to carry the design loads associated with the structure and the resulting settlement. Typically, up to 10 mm of settlement is considered acceptable at the design verification load for insensitive buildings. The designer's expectations of an acceptable test result should be set out in advance of the works. The design verification load (DVL) referred to in SPERW is typically 40–50% of the anticipated or previously measured ultimate bearing capacity. The load a pile will actually carry in service is referred to as the representative action (F_{ep}) or specified working load (SWL). The DVL should account for the F_{ep} and any specific differences that may occur between the pile load test and the pile in service, for example pile downdrag or variations in final levels of the site. Working piles are typically loaded to the DVL plus 50% of their F_{ep} (proof load test). Preliminary piles may be further subject to increments of 25% of their F_{ep}. When calculating the anticipated static capacity, it should be remembered that the calculated capacity should be as realistic as possible and avoid the adoption of unnecessarily conservative soil parameters to allow specification of adequate loading systems. The equipment required to carry out the testing should be specifically designed for the purpose, include careful consideration of the peak load required and the potential for change should be made at an early stage of the process. Once the equipment is on site increases in load application beyond that first envisaged cannot be easily accommodated.

98.3 Bi-directional pile testing

98.3.1 What is bi-directional load testing?

The bi-directional static load test is an alternative to the top-down static load tests described above (England, 2008). The method varies from the top-down load tests in that the major component of the system is a purpose-built high-capacity jack (or jacks) cast in the pile length. The most common form of this type of system is referred to as the O-Cell, which is receiving increased use and acceptance. SPERW (2017) refers to the bi-directional method as another form of the maintained load test and it is included in BS EN ISO 22477-1:2018 which focuses on compressive static load testing.

The bi-directional O-Cell method of testing was originally developed by Professor Jorj Osterberg to load the pile from the pile base rather than from the head and more specifically for the evaluation of skin friction and end-bearing in rock sockets. On pressurising the jack(s), reaction is provided by the pile end-bearing capacity to mobilise the pile's skin resistance and *vice versa* until the capacity of either the jack or the upper or lower components of resistance are exceeded. More recently, individual jacks or multiple jacks have been installed at various levels within cast *in situ* piles and diaphragm walls (barrettes) to allow testing of different sections of the foundation length (Randolph, 2003; England, 2008; England and Cheesman, 2010).

98.3.2 How does bi-directional testing work?

The system typically works by incorporating a purpose-built, low friction jacking device within a cast *in situ* pile (**Figure 98.8**). The loading arrangement is attached to the reinforcing steel or other support structure to ensure precise location and depth (**Figure 98.9**). When pile construction is complete and the concrete has attained sufficient strength, the jack is then pressurised using a hydraulic pump with the pressure monitored by a pressure transducer attached to the hydraulic return line. The foundation is then effectively separated into two elements which are subjected to simultaneous loading.

During testing, the separation of the jack is monitored by displacement transducers (LVDTs or LVWDTs) mounted between the two faces of the jack. Telltales are also attached to the top of the jack that extend up to the head of the pile, which allow the compression of the pile shaft to be monitored and the location of the top and bottom of the jack(s) to be determined. The movement of the pile head is also monitored along with compression of the pile. On completion of testing, the jack(s) and the annulus around the jack(s) can be grouted up through the hydraulic circuit to allow the pile to be integrated into the structure as a working pile. The bi-directional cells come in various diameters from 180 mm to 870 mm, with capacities from 0.9 MN to 27 MN. Test loads as high as 320 MN have been applied using several bi-directional installed at the same level in a pile socketed in rock and up to 360MN mobilised in a barrette. (www.loadtest.com).

Figure 98.8 Schematic of single level O-Cell testing arrangement. Image supplied courtesy of Fugro Loadtest; all rights reserved

Figure 98.9 Insertion of the O-Cell in the reinforcing cage of a diaphragm wall. Photograph courtesy of Fugro Loadtest; all rights reserved

98.3.3 The advantages and disadvantages of bi-directional testing

The bi-directional type of load test has several obvious advantages over static top-down methods. The systems require no large surface reaction such as kentledge or anchored steel reaction frames thus reducing space requirements, set-up time and transportation costs. The system is also safer with the loads being applied at depth. There are also reported cost savings with bi-directional load testing being comparable in cost with top-down static load tests at 5–10 MN but then becoming much more cost effective at higher loads. There is also the potential with the bi-directional cells to apply loads that cannot be achieved by top-down reaction or kentledge systems. Although the design of bi-directional cells lends itself to use in auger bored cast *in situ* piles it has also been successfully deployed in continuous flight auger piles (CFA), Fundex piles, drilled steel piles with rock sockets, steel and concrete driven piles and barrettes. Another advantage is to use a bi-directional test as a substitute for tension tests at high loads and push the pile up instead of attempting to pull the pile upwards from ground level.

One specific disadvantage associated with the bi-directional approach is that the jacking system needs to be pre-installed. This means it is not possible to select a random working pile for testing. The other disadvantage of the system is that the components installed within the pile are not recoverable after testing. This means that the bi-directional and some of the instrumentation is sacrificial which has a cost implication although when testing high loads the cost per unit load decreases with increasing magnitude (contrary to traditional top down load tests).

98.3.4 Interpreting the data from bi-directional testing

Typical results from a single O-Cell installation similar to that shown in **Figure 98.8** are shown in **Figures 98.10** and **98.11**. To estimate the load-displacement behaviour at the head of the pile several techniques can be used. The two most direct methods are to sum the measured responses and include the additional elastic shortening or alternatively model the response of each of the pile elements using Cemsolve (Fleming, 1992) and then add together the geotechnical behaviour and corresponding total anticipated elastic shortening.

The total load-displacement responses can be obtained by adding together the loads measured above and below the bi-directional cell (at the same level of displacement). Where one section has mobilised less displacement than the other, for instance in the case of the movement of the upper section (**Figure 98.10**), it may be necessary to extrapolate the obtained results using methods such as the Chin approach (Chin, 1972) for a single hyperbolic function or the Cemsolve approach (Fleming, 1992) to obtain the sum of the measured response as shown in **Figure 98.11**.

During the bi-directional loading some elastic compression is contained within the results. During top-down static loading where the full load is applied to the pile head additional elastic shortening would occur. This feature can be modelled and added to the results of bi-directional loading. Alternative approaches to analysis are discussed by England (2008).

Figure 98.10 Example of results from a typical bi-directional load test. Data supplied courtesy of Fugro Loadtest

Figure 98.11 Sum of the measured responses from a bi-directional load test. Data supplied courtesy of Fugro Loadtest

98.3.5 Standardisation and guidance for bi-directional testing

As a relatively new method of pile testing bi-directional testing has historically suffered from a lack of guidance to aid specification; for instance, it is not specifically mentioned in Eurocode 7. This has been addressed though in the UK by the second and third editions of the specification for piling and embedded retaining walls (ICE, 2017). The Federation of Piling Specialists (FPS, www.fps.org.uk) has also published guidance on use of the technique as part of a general pile testing guide. A more critical appraisal of the technique with case study comparisons can be found in a review of innovative pile testing techniques by Paikowsky (2004). Codification was produced in the US in 2018 with the publication of ASTM D8169/D8169M-18 and more recently as per other pile testing methods it as included in the informative annex of BS EN ISO 22477-1:2018. For Eurocode 7 purposes (and other codes and specifications where bi-directional testing is not specifically mentioned), it may be adequate to class the bi-directional test as a full-scale static load test as suggested in SPERW (2017).

98.3.6 Specifying a bi-directional test

Care needs to be taken when specifying systems where the jack is to be installed in a preliminary test as the optimum level will be where the frictional capacity upwards (above the cell) matches the available reaction below the cell. In an ideal test, the skin friction below the cell (downwards) is fully mobilised with sufficient of the end-bearing mobilised to allow characterisation before the ultimate skin friction above the cell (upwards) is mobilised. Where elements of the pile are not fully mobilised, back analysis to reveal ultimate capacity may be used (as discussed above) where sufficient movement has occurred.

If the required test loads cannot be accommodated at a single level due to limitations on pile diameter it may be appropriate to consider multiple installation levels so that the entire foundation element may be mobilised in turn during phased testing. A specific advantage of using multiple level jack installations is that it allows individual pile sections to be mobilised separately. Cells placed close to the pile tip should also give due regard to the quality of the base and concrete at this level with cells typically placed one to two diameters above the base.

As the bi-directional cell applies load via jacking systems it is possible to specify load increments, hold periods and loading rates as per static tests.

98.4 High strain dynamic pile testing

98.4.1 What is high strain dynamic testing?

The previously discussed load application techniques used to measure pile capacity typically have load application durations in terms of hours. Dynamic pile testing differs in that loading the pile occurs by a drop weight applying a very short duration impact (≈5 ms) load at the head of the pile. The impact of the weight produces a stress wave that travels down the pile. Where the movement of the pile is resisted or there is a change in impedance such as at the pile tip, a wave will be reflected back up the pile. Based upon comparing the waves travelling up and down the pile it is possible to assess the dynamic pile reaction. Through further analysis it is possible to derive the equivalent static pile load-displacement behaviour. Due to phenomena such as rate effects (including creep), excess pore water pressures, and inertia effects (due to acceleration) dynamic or rapid load test results might differ from what would be expected from an equivalent static load test (BS8004:2015). Further detailed guidance on dynamic pile test specification and analysis approached can be found in BS EN ISO 22477-4:2018.

98.4.2 Methods available for dynamic testing

Dynamic testing has the advantage that the main equipment required for monitoring the test is relatively compact and where the piles are being driven on site the normal pile-driving hammer can be used to produce the required stress wave. Piles are typically tested some time after installation which is referred to as re-strike. This involves the piling rig returning briefly to the test pile to deliver a series of hammer blows (typically five, depending on the approach taken). If the tests are being carried out on a cast *in situ* pile or are re-strike tests of driven piles where pile driving for installation is complete then it may be necessary to mobilise a separate drop weight system (**Figure 98.12**). Alternative dynamic testing methods are also available which may use dedicated testing equipment and utilize alternative approaches to testing and analysis e.g. the multi-blow dynamic testing technique referred to in BS EN ISO 22477-4:2018 and shown in Figure 98.12.

Figure 98.12 SIMBAT mini dynamic drop weight system (1000 kg hammer). Note pile head instrumentation installed on exposed concrete after local removal of temporary casing. Photograph courtesy of James Fisher Testing Services Ltd; all rights reserved

98.4.3 How does dynamic testing work?

Measurements are typically taken during the hammer impact from a pair of accelerometers and strain gauges that are attached above ground level to the pile head (BS EN ISO 22477-4:2018) either during or after driving (**Figure 98.13**). The response of this instrumentation is logged during and after the weight impact on a specifically designed logging and analysis device, for example the Pile Driving Analyser (PDA) produced by Pile Dynamics Inc. Data from the instrumentation are used to derive the force (F) applied to the pile by multiplying the measured strain by the cross-sectional rigidity of the pile and the velocity (v) from integration of the accelerometer readings.

The force measured by the strain gauges at the pile head due to the hammer impact is compared with the equivalent force derived from measurements made by the accelerometers (**Figure 98.14**). The data shown are for a re-strike test on a 250 mm square pre-cast concrete pile where the pile tip was 23.2 m below the strain gauges.

Figure 98.13 Installation of instrumentation (strain gauge and accelerometer) on a pre-cast concrete driven pile (pile hammer visible at top of image)

Figure 98.14 An example of measured pile stress wave data. Data provided by Technical Services Ltd

It is assumed, in stress wave theory, that both force and velocity are proportional as the wave travels through a material.

Where:

$$v = \frac{F}{Z} \tag{98.4}$$

$$\text{or } F = v \times Z \tag{98.5}$$

$$\text{the pile impedance } Z = \frac{E_p A_p}{c} \tag{98.6}$$

where E_p is the Young's modulus of the pile, A_p is the piles' cross-sectional area and c is the wave velocity in the pile which can be found based upon the stiffness and density of the pile (ρ_p):

$$c = \sqrt{\frac{E_p}{\rho_p}} \tag{98.7}$$

Where the movement of the pile is resisted, or there is a change in impedance such as at the pile tip, a wave will be reflected back up the pile (**Figure 98.15**). The total dynamic resistance of the pile (R) to the stress wave passing up and down the pile has been shown to equal the sum of the downward travelling force ($F_{(t_0)}$), force measured at maximum pile head velocity, ($v_{(t_0)}$) plus the upward travelling force that arrives back at the pile head ($F_{(t_0+2L/c)}$) at approximately $2L/c$ (where L is the pile length, **Figure 98.15**) after the initial peak load (Rausche *et al.*, 1985; Randolph, 2003).

$$R = \frac{F_{(t_0)} + F_{(t_0+2L/c)}}{2} + Z\left(v_{(t_0)} + v_{(t_0+2L/c)}\right) \tag{98.8}$$

Figure 98.15 Schematic of stress wave travel in a pile. Reproduced from Randolph (2003)

98.4.4 Advantages and disadvantages of dynamic testing

The advantages of dynamic testing come from the simplicity of the equipment and the speed of testing. If driven piles are being tested it is typical to test at re-strike at some time after driving. The on-pile strain gauge and accelerometer are quickly bolted to the test pile (**Figure 98.13**) with logging undertaken on a small hand-held logging and analysis system (**Figure 98.16**). This means the system can be operated by a single operator with quick instrument installation causing little disruption to construction. As the piles can be tested during installation many working piles can also be tested without concerns over testing altering working performance. This data collection process has been improved recently with the development of wireless instrumentation and the option of replacing strain gauge load determination with direct pile head load cell readings and direct measurement of pile head displacement with the use of high-speed optical scanning theodolites.

If the piles are cast *in situ* piles or are driven piles tested after the original pile driving rig has been demobilised then it is possible to mobilise a drop weight system. Tests up to 36 MN have been undertaken by dropping 36 tonne masses. Typical loads achieved are 1 MN for 1000–1500 kg drop weight systems (typically 1.5–2% of the applied load is required) and 3 MN for 4000 kg drop weight systems. The ability to produce large loads with limited drop mass significantly reduces cost (**Figure 98.12**).

Other benefits associated with the nature of the test mean that it may be possible to verify the integrity of the pile during normal testing (See Chapter 97). Through analysis it is also possible to investigate the distribution of resistance down the pile and the split of base to shaft resistance without the need for additional instrumentation as is required in techniques where only pile head measurements are made.

Due to the nature of using the test during pile driving the test only gives an indication of the pile resistance during the installation process. Depending on the soil type the capacity of the pile may change significantly after driving due to, for instance, the dissipation of pore pressures which may lead to increases in capacity with time, which is often referred to as set-up. This may lead to the need for re-strike testing at various times after driving to verify pile capacity. This is not a problem specific to dynamic testing but may need to be considered for any pile testing method where displacement piles are being tested.

Figure 98.16 Pile Driving Analyzer® capturing data during pile driving from wireless instrumentation. Photograph courtesy of Technical Services Ltd; all rights reserved

98.4.5 Interpretation of dynamic testing

It has previously been briefly explained how the dynamic pile resistance can be derived from pile head measurements during dynamic testing. It is then necessary to determine the ultimate static pile resistance from these measurements. Several methods are available to do this with the most common approaches based upon signal processing and numerical models (Holeyman, 1992).

98.4.5.1 CASE method

One of the earlier methods developed for signal processing is referred to as the CASE method (Rausche *et al.*, 1985). This simple technique may be used to quickly assess capacity in the field based upon an individual blow. It requires the determination of a damping force (R_d) which is assumed to be proportional to the pile tip velocity (v_{tip}) that can be removed from the total dynamic resistance to give the equivalent static resistance. Simplification lies in the assumption that dynamic pile resistance is proportional to pile tip velocity only.

$$R_d = JZv_{tip} = J\left(F_{(t_0)} + Zv_{(t_0)} - R\right) \tag{98.9}$$

The performance of this technique is dependent on the soil-dependent CASE damping factor (*J*). Suggested values for the damping factor are shown in **Table 98.6**. Depending on the exact time point at which dynamic pile capacity is determined when analyzing an individual blow some variation in the value of the damping parameters may occur. Further guidance is given in Rausche *et al.* (1985), Faehle E.A.P (2013) and BS EN ISO 22477-4:2018.

Soil type	Damping factor, J (s/m)
Sand	0.05–0.20
Silty sand/sandy silt	0.15–0.30
Silt	0.20–0.45
Silty clay/clayey silt	0.40–0.70
Clay	0.60–1.10

Table 98.6 Values for CASE damping coefficient. Data taken from Rausche *et al.* (1985)

Based upon the range of potential damping parameters shown in Table 98.6 it is recommended that the actual damping factors adopted are selected based upon the results of signal matching or experience of testing of similar piles in similar ground conditions.

As an alternative, the TNO closed form solution method includes both skin friction and pile tip related terms which results in different damping constants for both components. More detail on the TNO method can be found in Foeken *et al* (1996) and BS EN ISO 22477-4:2018.

98.4.5.2 Signal matching

The most common methods of analysing dynamic load tests presently used are based upon lumped parameter finite difference or finite element techniques where the pile is modelled as an assembly of interconnected masses with varying properties. These properties, predominantly soil parameters, are varied until computer-simulated pile head forces and velocities match those measured. Several computer packages have been developed that utilise this 'signal matching' method such as CAPWAP (Case pile wave analysis program), TNOWAVE and SIMBAT (Stain, 1992). Further detail on signal matching is given in BS EN ISO 22477-4:2018.

Common issues that may affect analysis are:

- insufficient pile settlement mobilised during testing to allow determination of ultimate pile capacity;
- variation in material properties or cross-section down the pile length;
- stresses locked in the pile as a result of pile driving.

It should be noted that these issues are not specific to dynamic testing and may cause problems to both static and rapid pile testing, especially where pile instrumentation is being used to supplement testing.

To allow the ultimate behaviour of a pile to be assessed from dynamic analysis it is necessary to mobilise adequate settlement in the pile and allow appropriate strain levels in the surrounding soil. Dynamic analysis based upon low settlement levels is likely to lead to a conservative assessment of pile performance, i.e. an underestimation of pile capacity. Where insufficient settlement is mobilised analysis is not compromised but reflects the mobilised soil resistance rather than ultimate pile capacity. Unfortunately, in the case of rapid load testing insufficient settlement may render analysis difficult. The situation of low mobilisation may come about due to ignorance of the importance of mobilising adequate settlement but more commonly it is a result of specifying rigs that are unable to apply the required load levels. Another reason for low mobilisation may be a limit on load/stress levels due to concerns over pile damage.

As dynamic test analysis is dependent on knowledge of the pile cross-sectional stiffness it is necessary to accurately know the cross-sectional profile of the pile and the stiffness of the pile material. These properties may be easy to determine for a steel or pre-cast concrete pile that is constructed to tight size tolerances and is cured in a factory environment. Where cast *in situ* piles are formed the profile of the pile may be less certain or complex, for instance in the case of a cast *in situ* screw pile or under-reamed piles. There are also material issues, i.e. how does the inclusion of reinforcement affect the pile stiffness and does site-cured concrete have the same properties as laboratory-cured samples? Where piles have complex known geometry this can easily be incorporated in dynamic analysis if as mentioned the geometry is known. Where less information is available for the test, for instance in the case of pile re-use assessment where there is little information about the length, cross-section and quality of historic piles (Butcher *et al*., 2006), dynamic testing may not be appropriate.

98.4.5.3 Multi-blow analysis

Further detailed guidance on the analysis of multi-blow testing can be found in BS EN ISO 22477-4:2018. This approach differs in that the aim is to determine a site-specific damping and soil dependent factors which reduces measured dynamic resistance to static resistance for a series of blows. The method consists of applying multiple blows of different drop height to the top of an instrumented pile, such that both high and low pile head displacements and hence pile velocities, are produced. Strain and acceleration are measured using sensors attached to the pile as described above with the pile head displacement signal measured using a high-speed optical scanning theodolite. Total dynamic pile resistance is determined in a similar manner to that described above (and described in more detail in BS EN ISO 22477-4:2018) and this resistance is compared for the different drop heights to determine the soil or site-specific correction factors due to controls such as soil dependent rate effects. The dynamic resistance is then corrected on this basis to obtain a static equivalent pile resistance. As per other dynamic or rapid methods this approach cannot capture the long-term behaviour of a pile where consolidation, pore pressure dissipation or creep will change or affect pile capacity with time.

98.4.6 Standardisation of dynamic testing

The dynamic pile testing method has been in common use for many years with BS EN ISO 22477-4:2018 giving useful recent coverage. In the UK, it is well documented in SPERW (2017) and the FPS general pile testing guide (FPS, 2007). SPERW (2017) also refers to both dynamic and rapid load pile testing as being well established. In the US, there is an ASTM standard test method for high-strain dynamic testing of piles (ASTM D4945-17). One common thread throughout the guidance documents is the need for experience of the dynamic load testing in similar soils and on similar pile types (EC7). Where such experience does not exist, or certain ground conditions prevail such as fine-grained and laminated soils it is recommended that the tests are calibrated against a site-specific static test (SPERW, 2017).

98.4.7 Specifying a dynamic test

It is important that the installation equipment being used for dynamic pile testing is first capable of driving the piles to the required depth or capacity and that it can mobilise the pile adequately during testing. Where time allows, details of the piles being driven and the driving equipment should be analysed prior to installation by the dynamic pile testing specialist. This wave equation analysis can be used to assess the ability of the proposed driving system to install the pile to the required capacity and desired penetration without exceeding allowable driving stresses in the pile material.

The other important decision to be made during dynamic testing is when to test, due to set-up and change of capacity with time. As mentioned, testing can be undertaken at the end of initial pile installation and through re-strike at any time after installation.

The piles to be tested dynamically should be designed to resist the driving stresses. They should also be designed to have at least 300 mm sticking up above ground at the end of pile testing to avoid sensor damage. To allow sensor installation it is important that there is adequate working room near the pile heads for sensor mounts to be installed prior to lifting piles. It is normal practice to have at least 1.5 diameters stickup above any attached instrumentation as a minimum. For cast *in situ* piles, it may be necessary to include an extension to the top of the pile to approximately three pile diameters above ground level and at least 100 mm above the existing pile reinforcement. This may be achieved by incorporating a section of casing above the pile head and filling with concrete while pouring the main pile concrete. Once the concrete has set windows may be cut in the casing to allow instruments to be attached to the concrete (**Figure 98.12**). Instrumentation is typically installed at a minimum of 1.5 to 2 pile diameters below the pile head. It is noted that dynamic testing and analysis is a relatively specialist area of pile testing and it is recommended that further guidance on appropriate analysis techniques and testing methodology should be sought through early engagement with a specialist pile testing contractor.

98.5 Rapid load testing

98.5.1 What is rapid load testing?

The main difference between rapid load pile testing and static and dynamic testing is the duration of application of load. A rapid load test usually has a load duration of 90–250 ms which is approximately 30 to 40 times that of a dynamic pile load test (ASTM D7383-19; Holscher and van Tol, 2009). The duration of the loading has been designed such that piles less than 40 m in length remain in compression throughout the loading event resulting in negligible stress wave effects (BS EN ISO 22477-10:2016) and simpler analysis (**Figure 98.17**). Generally, stress wave analysis cannot be utilised with this type of test.

Figure 98.17 Measured and calculated results from a 3000 kN Statnamic load cycle. Reproduced from Brown and Hyde (2006)

98.5.2 Methods available for rapid load testing

Several different techniques are available to produce rapid loading events. Statnamic testing works by rapid burning of a solid fuel that produces gas in a pressure chamber (**Figure 98.18**). The venting of this gas is used to accelerate a mass upward that in turn imparts a load on to the foundation pile below the Statnamic device. The load is applied and removed smoothly through the controlled venting of the gas. Alternatives to this approach typically take the form of a drop mass system where the load duration is increased through various cushioning techniques, for instance a spring or springs in the pseudo static pile load tester (Schellingerhout and Revoort, 1996) and spring hammer systems (Matsuzawa *et al*., 2008) and specialised cushioning in the hybrid dynamic test (Miyasaka *et al*., 2008) or tunable cushioning in the StatRapid system (Middendorp *et al*, 2013).

Figure 98.18 Statnamic pressure chamber with load cell at its base shown mounted on a test pile

98.5.3 What are the advantages and disadvantages of rapid load testing?

The main advantage of rapid load testing devices comes about as a result of the method of load application. For example, in the rapid load testing method known as Statnamic a reaction mass is accelerated upwards resulting in a load application of 3.5 MN from 18 tonne. Typically, for Statnamic testing the reaction mass is only 5% of the equivalent static load reaction mass required to produce the same load. This results in rapid load testing devices being significantly smaller and lighter than static equivalent set-ups. This means that they can be used on sites with space restrictions, are easier to mobilise and quicker to set up and test. For example, a 3 MN Statnamic rig (**Figure 98.19**) can be mobilised with one articulated truck and a 70 tonne crane. If the rig incorporates a hydraulic catch mechanism (catches the reaction mass) it can test up to 10 individual piles a day or allow multiple load cycles to be carried out on an individual pile with minutes between cycles.

Typically disadvantages of rapid load testing methods stem from the rapid nature of the load application which may result in piles displacing at rates measured in metres per second resulting in very high strain rates in the soil (**Figure 98.17**). In fine-grained soils, such as clay, such high strain rates have been shown to significantly increase the strength of the clay (Brown & Hyde, 2008). This may result in pile ultimate capacity up to three times the measured or predicted static pile capacity (Holeyman, 1992). This effect appears to become more pronounced as the liquidity index of a clay increases (Robinson 2019 & Robinson *et al.,* 2023). It is important to be aware of this effect to avoid specifying rapid load testing devices that are incapable of adequately mobilising a pile.

Another major drawback is that certain techniques such as Statnamic may operate over a very discrete loading range; for instance, a 3 MN Statnamic rig may apply loads as high as 3.5 MN but would not be able to apply loads below 700 kN.

98.5.4 How does StatRapid work?

Historically in Europe Statnamic was the most widely form of rapid load testing used but this has been overtaken more recently by StatRapid testing. As StatRapid is the most widely deployed method of rapid load testing in Europe it is described in more detail (**Figure 98.19**). The StatRapid test method uses a drop weight to axially load the pile by dropping a mass on to a specifically designed polymer type cushion which has modular elements in terms of the number of spring elements and stiffness of each spring element that make up the cushion. This allows optimsation of the loading event which is numerically modelled prior to testing to choose the spring modules that control the load pulse or “push” applied and to maintain pile compression throughout the loading event. The drop mass or weight is guided within a frame which includes hydraulics for lifting and catching the weight after impact to avoid multiple uncontrolled impacts on the pile. The test mass is also modular and weights can be added or removed in order to optimize the setup for each pile, depending on soil conditions and on the required test load.

Statnamic varies from StatRapid in that load is applied vertically down a pile by burning a rapid-burning fuel which produces significant volumes of gas within a piston which accelerates reaction mass upwards. The duration and shape of the load pulse (**Figure 98.17**) are controlled by venting of the gas. The reaction mass is forced upwards and is caught by either gravel in the case of large load tests 20–60 MN or by hydraulic catch mechanisms for smaller rigs (3–20 MN, **Figure 98.20**). As Statnamic load application is independent of gravity the device can easily be used to test raked piles and has been deployed horizontally to test piles laterally and simulate impacts on structures.

During rapid load tests, load is measured directly by a calibrated load cell mounted below the cushioning system or incorporated in the base of the combustion piston for Statnamic which sits on the pile head (**Figure 98.18**). Displacement is also measured directly using a non-contact system either by a photovoltaic sensor mounted in the piston (Stanamic) which is excited by a remote laser beam or the movement of targets mounted on the pile are recorded by means of a remote optical displacement tracking system such as a high-speed optical scanning theodolite. As the rapid load test event may induce stress waves in the ground it is important that the remote component of the measuring device is located far enough away from the pile to avoid disturbance during the duration of the loading event (Brown and Hyde, 2006). The recording of displacement–time history can then be differentiated once or twice to determine pile velocity and acceleration, both of which are required for test analysis.

It is also commonplace for the system to incorporate an accelerometer which can be used for direct recording of acceleration. The acceleration–time history can also be integrated to determine velocity which may be less ‘noisy’ than that found by differentiation and for a second time to verify optical or laser-based displacement measurements.

Figure 98.19 1.5 to 10 MN StatRapid rig with remote optical displacement tracking system for displacement measurement in the foreground. Photograph courtesy of Allnamics; all rights reserved

Figure 98.20 3.5 MN Statnamic rig with hydraulic catch mechanism ready for testing

98.5.5 How to interpret the data

Rapid load tests must be analysed to remove both the effects of inertia associated with the pile/soil and strain rate-dependent response of the ground. The most common approach for this is referred to as the unloading point method (UPM).

Based on inspection of load-displacement curves from Statnamic tests Middendorp (2000) observed that during the unloading, the pile velocity reached zero which in coarse-grained soils corresponded approximately to the ultimate static resistance of the pile. This point (the unloading point, UP) was then used to determine a constant damping coefficient to correct the measured Statnamic data for the velocity-dependent resistance of the soil. Unfortunately, when this technique is applied to piles installed in fine-grained soils there is a tendency for the ultimate pile capacity to be significantly overpredicted. In order to correct for this effect, a series of soil-dependent average correction factors were developed by Paikowsky (2004) (**Table 98.7**) which the derived static load was then multiplied by to obtain a corrected UPM analysis (**Figure 98.21**). More recently, it has been proposed that much greater average correction factors In clay are required resulting in a μ value of 0.47 (Weaver and Rollins, 2010). BS EN ISO 22477-10:2016 gives similar correction factors based upon recent experience in Europe and as collated in Hölscher *et al* (2011)

In response to the known shortcomings of the UPM technique, a series of alternative methods have been developed which have their basis in the velocity or strain rate-dependent behaviour of soils. One nonlinear velocity-dependent method of analysis was developed by Brown (2004). The main conceptual difference from UPM is that the method is reliant on the user input of soil-specific rate parameters:

$$F_u = \frac{F_{STN} - Ma}{1 + \left(\frac{F_{STN}}{F_{STNpeak}}\right)\alpha\left(\frac{\Delta v}{v_0}\right)^{\beta} - \left(\frac{F_{STN}}{F_{STNpeak}}\right)\alpha\left(\frac{v_{min}}{v_0}\right)^{\beta}} \qquad (98.10)$$

where F_u is the derived static pile resistance, F_{STN} is the measured Statnamic load, Ma is the pile inertia, Δv is the pile's velocity relative to the soil and v_{min} is the velocity of the static CRP pile test used to define the soil-specific rate parameters α and β (β is normally set to 0.2 in clay). Both Δv and v_{min} are normalised by v_0 which is assumed to be 1 m/s. This form of analysis has evolved from the analysis of dynamic tests where the majority of the pile capacity is developed through skin friction (Randolph, 2003). The value of α may be selected based on the tentative relationship with plasticity index (PI) as proposed by Powell and Brown (2006) and Brown and Powell (2013):

$$\alpha = 0.03PI(\%) + 0.5. \qquad (98.11)$$

More recent work looking at rate effects and influencing factors in clays using high speed triaxial testing (Robinson 2019) would suggest that rate effects in clays are influenced by the state of the soil which could be determined based upon the insitu moisture content (Brown and Hyde, 2008) and the clay particle surface area. Better correlations were found using the liquidity Index (LI) and there appeared to be a strong correlation with the slope of the critical state line which itself is thought to be influenced by a clay's particle surface area (Muir Wood, 1990).

Soil type	Rate effect factor (μ)	FOS without μ	FOS with μ
Rock	0.96	2.0	2.0
Sand	0.91	2.1	2.0
Silt	0.69	2.8	2.0
Clay	0.65	3.0	2.0
Note: FOS = Factor of safety			

Table 98.7 Correction factors for UPM analysis. Information taken from Paikowsky (2004)

Figure 98.21 Performance of UPM correction of Statnamic data. The pile tests have been reset to zero settlement to allow comparison

One additional benefit of rapid load testing methods that incorporate hydraulic catching and lifting mechanisms is the ability to quickly recycle the testing equipment and apply multiple cycles of loading to a single pile. BS EN ISO 22477-10:2016 outlines how the UP method of analysis can be modified to accommodate multiple load cycles.

98.5.6 Guidance and standardisation of rapid load testing

Like bi-directional testing, rapid load pile testing has historically suffered from a lack of guidance to aid specification. This has again been addressed in the UK by SPERW (2017) and the FPS pile testing guide (FPS, 2007). In the US, there is an ASTM standard for rapid load testing of deep foundations (ASTM D7383-19) along with guidance produced for the federal and state highways agencies (McVay *et al*., 2003; Paikowsky, 2004). The Japanese Geotechnical Society has also produced a testing specification, an English language draft of which was published in 2000. A European testing standard within the Eurocode/ISO framework (which has been adopted by British Standards) was developed and published in 2016 (EN ISO 22477-10:2016). This document was initially outlined by Hoelscher *et al*., 2010 and followed the publication of Dutch guidance (Hoelscher et al., 2011) which contains a useful database of rapid load testing in different soils. The publication of EN ISO 22477-10:2016 makes a clear statement that rapid load testing is its own seperate entity and not a sub-set of dynamic pile testing. This standard is predominantly concerned with test definition and guidance on how to undertake testing to achieve the best results but it does go onto outline potential methods of test analysis based upon the UPM approach for single and multi-cycle test. EN ISO 22477-10:2016 does not consider model or partial factors for design as these were considered the remit of an evolved Eurocode 7. Inspection of the evolved Eurocode 7 as part of public consultation would suggest that both Rapid and Dynamic load testing of piles will see greater independent recognition and guidance on how these techniques can be utilized in a Eurocode design framework in the future.

98.5.7 Specifying a rapid load test

Prior to specifying a rapid load test it is necessary to have access to the results of geotechnical investigation and laboratory testing to allow selection of adequate testing equipment and loading levels. This though should really be the case prior to undertaking any type of pile test. The encountered ground will also influence the analysis approach adopted. For analysis of tests in fine-grained soils, ideally regular determination of moisture content and plastic and liquid limits should be undertaken down to at least the depth of the toe of the pile.

In granular soils and piles installed in rock, the applied loads will be similar to the anticipated ultimate pile capacity with additional loading capacity required for the inertial forces generated by the pile and soil/rock. In addition to the inertial resistance, load testing devices used in fine-grained soils such as clays will need to be capable of applying additional loads due to strain rate effects. For example, in clay the ultimate capacity measured during rapid loading may be 1.8 times to 3.5 times greater than the static capacity or higher (Brown and Powell, 2013, **Figure 98.22**). The need to apply such high loads should be considered when designing and specifying test piles. Further detailed guidance can be found in BSEN ISO22477-10 Testing of piles: rapid load testing. There is also further useful commentary in SPERW (2017).

Figure 98.22 Cycles of Statnamic loading prior to analysis compared with CRP and MLT static tests (see Figure 98.1). Reproduced from Brown et al. (2006)

98.6 Pile testing safety

98.6.1 Why do we need to be careful?

The testing of piles poses a safety hazard for several reasons. The most obvious of these are the high tensile and compressive loads generated in both the pile and reaction system during testing. Failure in any part of the system may lead to rapid release of energy and collapse of the testing arrangement. Other issues may arise due to the specific reaction system: for instance, kentledge relies on placing weights above a supporting frame, this frame needs to be stable throughout erection and testing thus requires appropriate structural design and checks on bearing capacity.

98.6.2 Good practice

As relatively high loads are applied over the relatively small area of the pile head it is important that all attempts are made to reduce eccentricity of loading on the pile and in other areas of the reaction system such as connections to anchor piles. Excessive eccentricity can lead to large moments that the load or reaction system has not been designed for. Eccentricity can be minimised by accurate setting out of reaction piles relative to the test pile, correct alignment of the load column and proper mating of contact surfaces and levelling of reaction beams.

An obvious means of reducing risk to site and testing operatives is to restrict access to the load testing set-up and reduce the need to be close to the heavily loaded elements or reduce the need to work under kentledge. This can be achieved by restricting access only to relevant personnel. The need for direct access to the load column can also be reduced or removed through the use of computer-controlled loading jacks and displacement transducers. Fully automated testing systems have been developed which can be controlled remotely (England, 2002) although it is generally advised that tests should not be left unattended whilst loading.

Good practice can also be incorporated in design of the loading system. For instance, where the testing system uses reaction piles this should ideally incorporate a minimum of three anchor piles as systems based upon two reaction piles are significantly less stable. The use of a two-pile reaction system should be reserved for relatively low loading levels with high tolerance pile and reaction system positioning.

98.6.3 Loading and inspection

It is important that the maximum test load to be applied to the pile is agreed in advance of the test so that the test pile/pile cap and all associated elements of the load testing equipment can be designed and specified to apply the maximum test load safely. This information must be communicated to the relevant operatives and where possible relevant limiting capacities should be marked on individual elements of the testing equipment (see **Figures 98.4**, **98.5**, **98.6**).

It may be the intention during testing to take the pile to its ultimate capacity and cause significant permanent settlement. In some cases a pile's capacity during testing may exceed that anticipated prior to testing and require additional loads to prove ultimate behaviour. Although the goal may be to prove ultimate behaviour, this should never be used to justify exceeding the maximum test load the testing system has been designed for.

During the course of the load test the whole system should be monitored for eccentricities and excessive deflections with appropriate action taken if this occurs. Typical signs of distress may include:

- excessive deflection of the reaction beams;
- upward movement of tensile bars cast into reaction piles;
- horizontal deflection of the reaction system;
- movement of kentledge;
- difficulty in maintaining test loads.

Further key safety issue guidance is given in FPS Pile Testing Guide (FPS, 2006).

98.7 Simple overview of pile testing methods

Selection of the most appropriate pile testing technique will vary from one situation to the next. Criticism is often levelled at the different techniques for their shortcomings whereas any form of pile test (if correctly undertaken and analysed) should be viewed as a tool to give confidence in the design assumptions made, improve efficiency in construction and verify as-built performance. For instance, where a static load test identifies problems with trial pile performance during working pile construction, rapid or dynamic testing techniques can be rapidly deployed to assess the effect on the constructed working piles. This may then allow modifications to pile design and construction that can be used to mitigate problems during construction without significant effect on the construction programme. **Table 98.8** summarises some of the pros and cons of the methods described in this chapter and offers suggestions for potential deployment considerations. This table only serves as a quick summary and should not be used for definitive selection of a test. Decisions on the most appropriate testing method are influenced by many factors: for example, if there is a pile driving rig on site it will likely be more cost effective to undertake dynamic testing than to mobilise another technique. Definitive guidance should be sought from the specific providers of such techniques who can offer cost-effective and bespoke solutions both for deployment and subsequent analysis and interpretation.

Test Type	Advantages	Limitations	Potential deployment
Static	• Simple test • Simple & quick interpretation • Well understood & accepted	• Slow tests • Significant infrastructure (especially • High space requirements • Safety concerns with increasing loads	• Lower loading (<20 MN) • Low pile numbers • Trial piles/limited number of working
Bi-directional	• Capable of very high test loads (higher than other techniques) • Low infrastructure • Low space requirement • Cost effective as load magnitude increases	• Pile for testing needs to be preselected • Less experience • Analysis needs to take account of • Specialised analysis and interpretation	• Medium to very high loading (2–320 MN) • Low pile numbers • Trial piles/limited working piles
Dynamic	• Low infrastructure • Low space requirement • Fast tests • Quick repeat testing • Mature technique	• Pile damage may be a concern • Perceived reliability: Intermediate • Tests may be influenced by pile • Specialised analysis and interpretation	• Medium to high loading (1–35 MN) • Low to high pile numbers • Trial and working piles • Where pile driving equipment already on site
Rapid	• Low infrastructure • Low space requirement • Rapid testing • Quick repeat testing	• Less experience • Analysis techniques still evolving • Less case study experience • Availability of high-capacity	• Medium to high loading (0.6–40 MN) • Medium to high pile numbers • Trial and working piles • Quality control of working piles • Problems with working pile performance

Table 98.8 Summary of pile test characteristics and potential deployment criteria

98.8 Acknowledgements

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American Standard Testing Methods. Standard test method for deep foundations under static axial compressive load. ASTM D1143/ D1143M-20.

American Standard Testing Methods. Standard test method for high-strain dynamic testing of piles. ASTM D4945-17.

American Standard Testing Methods. Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial

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98.9.1 Useful websites

Federation of Piling Specialists (FPS); www.fps.org.uk

Allnamics; allnamics.com

Pile Dynamics Inc. and GRL Engineers Inc.; www.pile.com

Fugro Load Test; www.loadtest.co.uk

It is recommended this chapter is read in conjunction with

- Chapter 8 *Health and safety in geotechnical engineering*
- Chapter 22 *Behaviour of single piles under vertical loads*
- Chapter 54 *Single piles*
- Chapter 81 *Types of bearing piles*
- Chapter 82 *Piling problems*
- Chapter 97 *Pile integrity testing*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set out in Section 4 *Site investigation.*