AS 2159 Supplement 1—1996

Piling—Design and installation—Guidelines

(Supplement to AS 2159—1995)

This Australian Standard was prepared by Committee CE/18, Piling. It was approved on behalf of the Council of Standards Australia on 14 December 1995 and published on 5 March 1996.

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(Supplement to AS 2159—1995)

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PREFACE

These Guidelines have been produced by the Standards Australia Committee CE/18 on Piling.

The material contained in these Guidelines was assembled during the preparation of AS 2159—1995, but was not considered to be of a mandatory nature and hence not appropriate for inclusion in the Standard. However, the Committee considered that the material would be helpful to designers and constructors and agreed that the material be published in this separate document.

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STANDARDS AUSTRALIA

Piling—Design and installation—Guidelines (Supplement to AS 2159—1995)

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE These Guidelines provide information on pile design and installation in relation to AS 2159.

1.2 REFERENCED DOCUMENTS The following documents are referred to in this document:

AS	
1604	Timber—Preservative-treated—Sawn and round
1726	Geotechnical site investigations
2159	Piling—Design and installation
2239	Galvanic (sacrificial) anodes for cathodic protection
2312	Guide to the protection of iron and steel against exterior atmospheric corrosion
3600	Concrete structures

ASTM	
D 3689	Method of Testing Individual Piles Under Static Axial Tensile Load
D 3966	Method of Testing Piles Under Lateral Loads

SECTION 2 SITE INVESTIGATIONS

2.1 INTRODUCTORY NOTE An essential prerequisite to any piling installation is an adequate site investigation. An outline of methods of investigation is given in AS 1726, and the information and techniques presented in that document should be followed.

The objective of this Section is to draw attention to requirements and procedures specifically related to piled foundations. These will be dependent in part on the subsurface conditions and also on the particular pile system to be used. Attention is drawn to the fact that the level of information required for general design purposes may be different from that required for construction.

For preliminary design, sufficient information is required to define the range of likely conditions, the soil/rock parameters on which the design is to be based and the selection of the most suitable pile system. Information is required regarding soil/rock strengths, compressibility and variation across the site. Ground water levels are to be determined.

For detailed design, tendering and construction purposes, a more intensive investigation may be appropriate, sufficient to allow reliable estimation by designers or contractors and to allow confident bidding and planning of work by the organization responsible for the installation. Factors such as level and inflow of ground water, caving conditions, penetrability of hard layers and range of depths may not be particularly relevant to design (sometimes they are), but may be of crucial importance during tendering and construction. For certain pile systems (e.g. bored or continuous auger piles in sands), which do not depend on recognition during construction (e.g. installation resistance, cuttings returned), site investigation may be the basis on which pile construction is controlled.

2.2 PRELIMINARY INVESTIGATIONS An investigation of the ground should be carried out by competent and experienced persons. Borings should reach depths adequate to explore the nature of the soil, both around and beneath the proposed piles, including all strata likely to contribute significantly to settlement. In cohesive soils, undisturbed samples should be taken from the borings and tested for strength, compressibility and other characteristics, to provide information on the carrying capacities of the soils at various depths, so that a preliminary estimate of the length and spacing of the piles may be made. Penetration tests and tests of disturbed samples are of value in assessing the variations in the ground conditions when boring through granular soils.

It is important that the nature and occurrence of ground water should be investigated. If the standing levels vary from stratum to stratum, or if there is a watertable gradient between boreholes, this should be noted. Ground water or soil may contain harmful constituents in amounts sufficient to cause damage to portland cement concrete or buried metals. Chemical analyses of samples of the ground water and soil should be undertaken, to assess the necessity for special precautions.

The preliminary investigation should include a careful appraisal of nearby structures and substructures, including the types and layout of all services near and through the site. The choice of pile may be influenced by the effects which its installation would produce on these structures and services. The appraisal should include examination from records or by trial holes of the nature of nearby foundations and any evidence of past settlement, subsidence or slips should be noted.

2.3 INVESTIGATION TECHNIQUES Site investigation for piling will almost always involve a number of boreholes and in situ tests. The following procedures are recommended for appropriate situations:

(a) *Boreholes* Bores may be drilled by auger methods in non-caving soils and by rotary methods with casing or mud support in caving soils. Bores should incorporate regular sampling or in situ testing of properties conducted at depth intervals not greater than 2 m, and preferably at 1.0–1.5 m intervals around and immediately below the founding zone. Bores should be taken to sufficient depth to 'prove' the founding layer and to investigate any weaker underlying zones which may affect pile performance. Standard penetration tests (SPT) (see AS 1726), are recommended in sands and may also be appropriate in clays. Alternatively in clays, undisturbed sampling and laboratory strength testing can be utilized.

The *RL* of each borehole should be recorded on the borehole logs.

(b) *Continuous penetration test* (see AS 1726) This test has advantages in low cost and in the provision of a continuous record of soil strength. The technique is ideal for situations where piles are founded in soils, provided that the rig has sufficient capacity to ensure penetration to sufficient distance below the founding level to prove its adequacy. The test will normally reach refusal on rock and may also refuse in very dense or thick sand or gravel layers or on isolated floaters. There may be a need for supplementary drilling and coring, to prove rock strength or soil consistency and continuity.

(c) *Diamond core drilling* Continuous core drilling of rock is appropriate where piles are to be founded on or in rock strata. The extent of this will be dependent on the pile type and loads.

(d) *Initial survey* This should incorporate a study of already available information regarding local geology, site history (has it been filled?), type, depth and performance of piles on nearby sites. The initial survey may make use of aerial photographs.

(e) *Trial drilling and piling* Conventional investigation techniques do not always provide information relevant to construction, e.g. drillability of certain strata, ground water level and inflow, potential for caving and need for casing and penetration of driven piles. In such circumstances, it may be appropriate to undertake trial drilling or piling with construction equipment of similar type, size and capacity to that proposed for the eventual construction.

Such drilling or piling may be followed by testing, to determine ultimate load on pile or load-settlement performance.

2.4 BORE FREQUENCY/SPACING The number of boreholes or tests on any site will be a function of site variability, the pile type proposed and, to some extent, the owner's requirements in regard to accuracy of cost estimates, fixing of contract levels and 'confidence' in contractor bidding.

2.5 TESTS OF SOIL/GROUND WATER AGGRESSIVENESS The investigation should take account of possible aggressive attack on steel and concrete and appropriate testing should be carried out where this is suspected. Tests of soil or ground water samples for measurement of Ph, chlorides and sulfates, and also of soil resistivity should be considered for assessing aggressive or corrosive attack.

Exposure classifications for concrete and steel piles, based on soil and groundwater chemistry are given in AS 2159, Tables 6.1 and 6.3.

2.6 COMMENTS RELATING TO SPECIFIC PILE TYPES

2.6.1 Driven piles

2.6.1.1 *General* Driven piles are frequently 'designed' on the basis of soil/rock properties, but their installation is usually controlled by dynamic measurements taken during installation. The geotechnical design of the required length of driven piles is therefore only an initial estimate.

2.6.1.2 *Driven cast-in-place piles in soil* This pile system can readily cope with variations in pile depth (within the range of the installation equipment) and a lower level of information is required than with most other pile types. Installation is almost always controlled by installation resistance, and site investigation is used primarily to estimate pile suitability, expected depths and installation conditions. For piles in soil, CPTs or bores with SPTs or undisturbed clay samples are usually adequate. Where piles are founded in clays, strength testing of undisturbed samples is advantageous.

2.6.1.3 *Driven cast-in-place piles founded in rock* The main requirement of investigation is to confirm the presence and condition of rock. Pile installation is controlled mainly by installation resistance, and only broad information regarding rock strength is necessary, since for most piles, penetration into the rock will only vary between zero and a maximum of about 2 m.

2.6.1.4 *Driven pre-formed piles in soil* A slightly more intensive level of investigation is required than for cast-in-place piles, mainly to facilitate accurate pre-ordering of materials. Pile installation is normally controlled by measurement during installation, with site investigation and geotechnical design serving primarily as a means of estimating pile lengths in advance.. Boreholes with SPTs, or samples with laboratory testing are also appropriate. For unusual situations, or where accurate pre-ordering of piles is required, trial piling could be advantageous.

2.6.1.5 *Driven pre-formed piles to rock* As with cast in-place piles, the main requirement is to assess rock levels and the conditions through which the piles have to be driven. By comparison with driven cast-in-place piles, a higher intensity of investigation is required to accurately assess pile lengths to facilitate preordering of pile sections.

2.6.2 Bored piles The assessment of founding levels for bored piles in either soils or rock will be controlled by geotechnical information relating to soil/rock strength and the appropriate bearing capacity or shaft adhesion. Some of this information, sufficient at least to fix the design parameters, will need to be determined in advance of the piling construction. Supplementary proving work may also be required during construction and may consist of inspection of cuttings, inspection of the pile base, or proving holes drilled below the pile base to confirm adequacy and continuity in accordance with the design.

Investigation of bored piles should also provide information relevant to ground water, potential caving conditions, and the need for temporary or permanent casing.

Attention is drawn to the fact that small diameter boreholes may not always provide reliable information on drillability and ground water effects, and there may be advantages in drilling larger diameter bores to check these conditions.

Where bored piles are founded on rock, information on rock strength will need to be obtained by core drilling.

2.6.3 Continuous flight auger piles in soil Where these piles are installed without detailed monitoring, the installation has to be controlled by the results of site investigation. This leads to a much increased need for investigation in advance of piling, by comparison with other pile types.

Bores or tests should be taken to some depth below the anticipated founding level, to ensure adequacy of the founding layer.

Trial piling with installation of one or more piles followed by load testing to determine load-settlement performance and ultimate load could be advantageous in optimizing the design, particularly for larger projects as an alternative to a more conservative design approach.

2.6.4 Continuous flight auger piles on rock In this instance, the pile installation will generally feel the surface of the rock as an increase in resistance to penetration. A lesser level of investigation may be appropriate than for similar piles founded in soil, but should be adequate to assess rock strengths and appropriate design parameters for end-bearing and shaft adhesion.

SECTION 3 REFERENCES FOR DESIGN CALCULATIONS

3.1 GENERAL

This Section contains a summary of key references which are considered to offer a reasonable basis for design calculations. In most cases, it is necessary to assess the sensitivity of a design by use of alternative methods of calculation; therefore, this Section is divided into seven main parts. The first part lists general books on pile and foundation design and other key references that span several aspects of pile design. Subsequent parts list references that address specific aspects of pile design in turn such as —

(a) axial capacity of single piles and pile groups;

(b) dynamic analysis of piles (driving and dynamic testing);

(c) settlement of single piles, pile groups and piled rafts;

(d) lateral response of piles;

(e) dynamic response of single piles and pile groups; and

(f) miscellaneous topics including effects of external soil movement, torsional loading, cyclic loading, buckling and flutter of piles.

It must be emphasised that the bibliography is by no means complete, but has been deliberately kept concise.

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SECTION 4 DURABILITY

4.1 CONCRETE PILES

4.1.1 General Good quality concrete has satisfactory durability for many purposes, but for some applications consideration should be given to the effect of certain aggressive agents on concrete below ground or in sea or fresh water. The extent to which precautions are required, depends considerably on the particular site conditions, so that detailed recommendations cannot be given. Much will depend on first hand knowledge of the ground conditions surrounding the concrete; where there is any doubt, a ground investigation should be undertaken together with a chemical analysis of the soil and ground water. Particular care is needed with old industrial sites, landfill and mine sites.

4.1.2 Sulfate attack Sulfate salts may occur in solid form in the natural soil, contaminated ground, fill or in dissolved form in ground waters or sea water. The rate of attack for a particular type of cement depends on the concentration of the solution, the ground water conditions and the permeability of the soil. The type of sulfate and the chemistry of the ground can significantly affect the rate of attack. For example, the chemistry of sea water leads to a lower risk of sulfate attack damage due to an absence of expansion damage in the presence of both sulfates and chlorides in high concentrations. The method of construction also has a significant effect on the rate of attack, as it determines the age at which the surface is exposed to a sulfate environment.

To resist sulfate attack it is essential that concrete is dense and well compacted. Low concrete permeability and choice of cement type is more important than high characteristic strength. Soil permeability is an important factor and the ease with which the contaminated ground water can move around and be replaced is all important. Where piles are installed in an impermeable clay soil, acid or sulfate attack only penetrates the concrete to such a small extent that the incorporation of a few centimetres of dense ‘sacrificial concrete’ will obviate the need for special cements.

Improved sulfate resistance can be achieved by using sulfate resisting cement.

4.1.3 Acid attack Well compacted, impermeable concrete, particularly if made with limestone aggregates, is resistant to low concentrations of acid, but strong solutions will attack concrete made with all types of cement. Pile jacketing or use of an alternative pile material may be required in such cases. Creek and swamp water usually contains organic acids from plant decay and free carbon dioxide which may slowly dissolve cement from any concrete surface against which it flows. Porous concrete may be significantly affected and therefore benefit from a protective membrane, but dense uncracked concrete will have less need for protection.

Acidic soil can occur either naturally (e.g. humic and carbonic acids), or due to industrial, mining or domestic contamination.

Resistance to water permeability and choice of cement type improve acid resistance. The use of slag and flyash additives or the use of silica fume are highly beneficial in resisting acidic attack.

The chemistry of the soil and ground water and the tendency for the Ph to change over the service life of the pile should be carefully assessed. Organic activity and bacterial activity can influence the dynamics of Ph over a period of time.

4.1.4 Chloride content Whenever there are chlorides in concrete above a threshold concentration, there is a risk of corrosion to embedded steel.

It is recommended that the total chloride content of the concrete mix arising from the aggregate together with that from any admixtures and any other external sources should not exceed the limits given in AS 3600.

4.1.5 Corrosion of reinforcement Steel in concrete is normally stable due to the formation of an iron oxide film on the steel surface which occurs under alkaline conditions. Corrosion initiation can occur either due to depletion of the concrete alkalinity (e.g. acidic conditions or leaching of lime from the concrete), or by the presence of chlorides (which break down the passive iron oxide film), or by other means (e.g. stray current corrosion). The mechanisms leading to reinforcement corrosion damage are commonly modelled qualitatively as a two-step process known as corrosion initiation and corrosion propagation, described as follows:

(a) *Corrosion initiation* By assessing the severity of the service environment, the design process should account for the provision of adequate concrete quality and cover to reinforcement to ensure that corrosion initiation does not occur during the service life of the piles.

For example, in the case of a pile located in a marine environment, chlorides can diffuse through the cover zone of concrete, to initiate corrosion. The mechanisms of chloride transport through the cover zone of concrete are different for each exposure zone as follows:

(i) *Submerged zone*—waterborne chlorides are transported due to a hydrostatic pressure gradient.

(ii) *Splash zone*—the wetting and drying effects of wave splash cause surface transport of chlorides, via capillary suction of chlorides, followed by ionic diffusion due to a concentration gradient.

(iii) *Atmospheric zone*—chlorides are deposited on the concrete surface either as sea water droplets or as aerosol. Chloride penetration then occurs as a result of ionic diffusion.

The threshold chloride content for predicting the risk of corrosion is commonly expressed in terms of either total chloride content, free chloride content, or the free chloride/hydroxide ion ratio. Due to laboratory requirements, total chloride content is usually measured to assess corrosion risk a conservative threshold limit of 0.06% (total weight of concrete) is given, although a range can be expected due to, e.g. cement type.

(b) *Corrosion propagation* Once the reinforcement has been depassivated, corrosion can be expected to propagate at a rate which depends on the availability of oxygen to complete the cathodic reaction and also the resistivity of the electrolyte (cover concrete). The resistivity of the cover concrete is chiefly a function of moisture content. A corrosion cell is set up with an adjacent area of passive reinforcement acting as a cathode where oxygen is reduced with the anodic dissolution of iron taking place at a small central anode area.

Since the volume of the product of corrosion exceeds the volume of the parent reinforcing steel, bursting pressures result in the subsequent cracking and spalling of the cover concrete.

By utilizing environmental severity data, chloride resistant concrete can be designed and specified to achieve a corrosion resistant service life. Where doubt exists, trial concrete mixes can be manufactured and tested for chloride resistance. Chloride resistance can be determined by imitating the service environment and, in the case of chloride ingress, a penetration coefficient can be ascertained, which can be realistically specified to achieve a corrosion-free life.

The protection of reinforcement depends upon the quality of the concrete, its compaction and impermeability as well as the amount of cover. The parts of a structure most susceptible to the corrosion of embedded steel are those exposed to intermittent wetting and drying, especially by sea and moorland water. Minimum cover requirements to reinforcement for various conditions of exposure and grades of concrete are given in AS 2159, Table 6.2.

4.1.6 Industrial waste tips Conditions found in industrial waste tips are generally the most difficult to deal with as far as the protection of concrete is concerned. Ground aggressiveness can range from ‘mild’ to ‘highly aggressive’, depending on the specific chemical composition of the tip. The ground water is sometimes acidic and sulfates may be present in high concentrations, particularly in ground near colliery waste tips.

The situation can be complicated by the presence of chlorides (if near a marine environment), and other aggressive media (such as certain types of industrial waste, e.g. ammonium nitrate). Aggression can be accelerated by bacterial action.

4.1.7 Unsuitable aggregates Certain aggregates in the presence of moisture are known to react with the soluble alkali content of the concrete, causing expansion and disruption of the concrete. The general solution to the problem is to exclude moisture and limit the total water soluble alkali content of the concrete to less than 3.0 kg/m^3.

Other aggressive agents that can attack concrete include magnesium salts, ammonium salts and specific fats and oils.

4.1.8 Frost attack While buried concrete is unlikely to be subject to frost attack, consideration should be given to the effect of freezing and thawing on any concrete partially exposed to the atmosphere. Where necessary, the entrainment of approximately 5% of air in the concrete is recommended to improve the frost resistance of concrete made with Portland cement and having a maximum aggregate size of 20 mm.

4.2 STEEL PILES

4.2.1 Introduction The corrosion of unprotected steel piles is a complex process, and although the mechanisms are generally understood, insufficient information is available to allow the reliable prediction of rates of corrosion for differing exposure conditions.

These notes are aimed at providing background information regarding the corrosion process and supplementing the requirements given in AS 2159, Section 6.3.

4.2.2 General Various studies of the corrosion of piles driven into undisturbed soil (and subsequently extracted), have led to the conclusion that—

(a) in-ground rates of corrosion observed have been much less than might have been expected from common experience or measurement of corrosion rates of buried steel specimens in shallow trenches in disturbed soil; and

(b) measurements of soil properties such as type, drainage, resistivity, Ph or chemical composition are not reliable indicators for assessing corrosiveness of steel piling.

There is increasing acceptance of the concept of a sacrificial corrosion allowance to provide for loss of section. The corrosion allowances given in the Table 6.4 of AS 2159 are probably conservative and should be varied if better information is available (e.g. from extraction and measurement of previous piles on a site).

4.2.3 Corrosion mechanisms

4.2.3.1 *General* The corrosion mechanism for steel is electrochemical, requiring the presence of moisture. Voltage to sustain the process results from the reduction of oxygen. There are three simultaneous processes, as follows:

(a) An anodic reaction, whereby iron is dissolved:

$$Fe \rightarrow Fe^{++} + 2E^{-}$$

(b) A cathodic reaction, where an equivalent amount of oxygen is reduced:

$$\frac{1}{2}\,O_2 + H_2O + 2E^{-} \rightarrow 2\,OH^{-}$$

$$\frac{1}{2}O_2 + H_2O + 2E^{-}\text{-}2OH^{-}$$

(c) An electric current is induced within the metal, completed by movement of ions within the electrolyte furnished by the liquid medium in contact with the metal.

The rate of corrosion will be the rate of slowest process and may be lessened by slowing any one of these processes. For example, an applied electrical potential 'cathodic protection' can prevent the anodic process, restricted oxygen access will limit the cathodic reaction and low electrical conductivity will limit the current flow in the solution.

Two problems that are relatively common in buried pipework do not appear to be relevant to steel piles. These are bacterial corrosion (it occurs with pipework but does not seem to occur significantly with piling) and stray currents (e.g. from electric rail systems). In the case of isolated piles, the voltage differential laterally across the pile is insufficient to cause problems.

4.2.3.2 *Piles in water* Piles which extend above the ground through sea or river water have been the subject of extensive studies, both of the corrosion of steel and the efficiency of corrosion prevention measures.

The protective effect of marine growth and corrosion products can be influenced by variations in salinity and temperature; given time, the former reduces the effect of the latter. An increase in temperature produces an increase in the rate of attack on steels (a rule of thumb is that the corrosion rate doubles every 10°C rise in temperature). At the same time, there may be a decrease in the rate of attack due to a reduced solubility of oxygen in the water. Temperature also affects the rate of marine growth with higher temperatures causing denser growth and accelerated activity.

The rate of corrosion appears to increase with water velocity, but pollution such as silt, oil or grease that floats on the surface and coats the metal often gives considerable protection. Water soluble pollution such as industrial effluents and acids may have the opposite effect. The protective nature of some rust layers arises from the barrier action which the rust presents to the passage of the reactants and products of the corrosion reaction. The gradually increasing protective nature of the rust layers on the extent of corrosion has been confirmed by corrosion data for steel piles used in a marine pier which show an average corrosion rate of 0.05 mm/year for the first 20 years and subsequent rates of about 0.025 mm/year (Ref. 4.4.1).

Corrosion rates vary along the length of piles and there is considerable variation between sites. The anodic corrosion peak is usually at the low water mark, wave action in the intertidal zone producing a protective, highly oxygenated cathode. Vigorous wave action, which removes marine growth and corrosion products (particularly where water velocity can suspend abrasive sand) will lead to high corrosion rates. Should data be required for a particular site, corrosion coupons are of little value as the differential aeration rate is not represented. The most practical method of obtaining data for a given site is to examine the published data or other records for similar sites.

Some values of corrosion rates found by Eadie and Kinson (Ref. 4.4.2), for unprotected situations are given in Table 4.1.

TABLE 4.1

CORROSION RATES ON CARBON STEEL PILES IN WATER

(After Eadie and Kinson (Ref.4.4.2))

Location	Exposure period (y_e)	Maximum corrosion rates per exposed face (μm/y_e)			
		Buried	Submerged	Intertidal	Splash
Port Adelaide* South Australia (originally coated)	52	30	50	40	70
Queenscliff* Victoria, Australia	17	—	70	55	110
Wrightsville† Beach, N. Carolina, USA	8	—	110	20	300
Mayport,† Florida, USA	5.5	—	160	30	530
Mobile Bay,† Alabama, USA	7	—	200	160	330
Dam Neck,† Virginia, USA (bare steel)	6	100	440	240	440
Dam Neck† Virginia, USA (coated)	6	76	6	44	110
La Costa Island† Caribbean, USA (bare steel)	5	46	420	320	425
Boston Harbour,* Massachusetts, USA	10	—	320	—	40
Lowestoft, UK* (coated waterside)	20	15	40	13	15
Port Kembla,* NSW, Australia	13	25	85	100	100

* Sheltered marine

† Exposed marine

NOTE: Corrosion rates for inland waterways would be approximately 50% of the above rates for sheltered conditions and the use of bare steel will normally be the most economic choice.

4.2.3.3 *Piles in soil* Of a number of studies made of in-ground corrosion rates, that of Oksaki (Ref. 4.4.3) is the most extensive. This involved 10 sites in Japan, in each of which 100 × 100 × 10 mm angle sections 15 m long were installed. Batches of three to five at each site were withdrawn after exposures of 2, 5 and 10 years. Site conditions comprised a variety of soil types, including reclaimed ground and fluctuating ground water tables. The main conclusions were as follows:

(a) No statistically significant correlations occurred between corrosion rate and any of the soil parameters measured at every site (depth, soil type, soil strength, pH, and resistivity).

(b) Slight but not statistically significant increase in corrosion at low pH values.

(c) Slightly higher corrosion rates in the top 2.5–3 m.

(d) No abnormal corrosion rates in filled ground (sands and silts but not industrial waste or domestic refuse).

Other studies have reported increased corrosion rates in saline ground water conditions and within the zone of ground water table fluctuations.

Arising out of these studies has developed the concept of ‘corrosion allowance’, which is the predicted loss of steel from each exposed face per year. The recommendations of various authors are in the range 5-20 µm per year but up to 30 µm per year in near surface areas which are actively cathodic.

Usually, though, corrosion data are not available for a given site, hence the site must be evaluated with reference to others where corrosion data have been measured. Some representative data for this purpose are given in Table 4.2.

4.2.4 Protective measures

4.2.4.1 *Piles in water* In river and marine situations, piles may require protection to achieve the intended service life.

Cathodic protection by an impressed (controlled), current is frequently used for major structures (e.g. oil platforms). However, it is only slightly effective in the tidal zone and totally ineffective in the splash and atmospheric zones.

Coatings may alternatively be used to provide protection, but in some cases may have limited life. Careful surface preparation is required before application. The following aspects should be remembered:

(a) Maintenance is usually impracticable in structures beneath ground level and below the tidal range.

(b) Paints which require a clean dry surface for their application will not be suitable for maintenance painting of steel between tides.

(c) Paint coatings may be damaged or removed during the driving of piles, particularly in a soil such as gravel, during handling of piles and by rubbing against guide railings during pitching and driving.

If there is a possibility that coating life may be insufficient, it may be good practice to allow for the possible use of cathodic protection in the future, by ensuring electrical continuity throughout the structure at the time of construction. The need for completing the system and for eventual start-up if any, can be determined by examining the structure at intervals.

Protection can also be provided by concrete encasement, beginning at the atmospheric area continuing down either to below low water level or to a level below the mud line.

For simple structures, where little maintenance is practicable (e.g. sheet piling), a combination of protective coating and sacrificial zinc or aluminium anodes may be used.

TABLE 4.2

CORROSION RATES OF UNCOATED STEEL PILES IN SOIL

Location	Exposure (Years)	Soil profile (Metres, description)	Soil chemistry	Depth and corrosion rates (Metres, µm/year)	
Burnley, Vic., Bank Yarra River	4.1	0–0.3 loose fill + clay 0.3–1, sand 1–2 sand + clay 2–4, clay 4–5.5, sand + clay 5.5–10 clay 10.8 rock	no obvious corrosive indications	0.8 3.8 7.0 9.9	29 27 24 3
Melbourne, Vic., Bank Yarra River	5.6	0–2 fill (silt) 2–3, clay 3–23 silt (sand lenses at 6, 12 and 18)	high organic content high salinity sulfate content high sulfate content sulfate reducing bacteria		27 below fill zone
West Melbourne, Vic., bank Moonee Ponds Creek	2.6	0–4, fill (gravel, sand, crushed stones) 4–15, silt	high organic content high salinity, sulfate content high sulfide content sulfate reducing bacteria	1.8 4.4 7.0 9.6 12.3 14.7	13 22 12 10 12 12
Hampton Park, Vic. Embankment of freeway bridge	4.0	0–8.6, fresh fill 8.6–15, natural soil	no obvious corrosion indications	no corrosion detected by ultrasonic means	
Murray Bridge, S.A. Bank Murray River	7.5	0– 18, clay 18-20 gravelly sand + limestone	sulfate reducing bacteria	0.7 1.1 10.7 11.1 20.5 20.9	52 18 0 3 3 5

NOTE: For more information see 'BHP Steel Piling', August 1973, p33.

4.2.4.2 *Piles in soil* The simplest and probably most economical approach is to provide for a 'corrosion allowance', determined in accordance with AS 2159, Table 6.4. Different allowances may need to be made over the length of the pile, possibly with higher allowance in the upper zone. Since this is frequently the area where stresses are highest, there may be economy in the use of welded-on, sacrificial sections.

Cathodic protection (see AS 2239) and protective coatings (see AS 2312) may also be used as appropriate. The general remarks given above are again applicable.

4.2.4.3 *Piles in the atmosphere* Where piles extend and are exposed above ground and water levels, protection, where required, should be provided by protective coating designed in accordance with AS 2312.

4.3 TIMBER PILES

4.3.1 Timber preservation If timber piles are employed without adequate preservation protection, the outer sapwood, which is non-durable in all timber species, should not be considered for the purpose of measurement of cross-sectional area, load-bearing capacity or permanency in any other regard.

Timber piling is subject to deterioration when in ground contact, due to decay and termite attack. When immersed in tidal salt water, it is further at risk of attack from marine boring organisms.

AS 1604 categorizes timber into groups of similar durability, based on the ability of the inner heartwood to withstand decay and termite attack in the critical ground contact situation.

The outer sapwood of all species is non-durable if untreated. However, if adequately impregnated with preservative, it is at least as durable as the untreated truewood of the most durable species.

A timber pile driven to below permanent watertable level will have an indefinite life even without preservative treatment. When any portion of a timber pile is above the watertable level however, the sapwood will be at risk of biological attack, regardless of species, unless treated with preservative. The truewood of some species will also be at risk.

Timber is particularly resistant to attack from chemically active ground water and is known to withstand a wide range of chemical environments. Depending on timber species used, the range of resistance to the chemical environment lies somewhere within the overall bounds from Ph 2 to 11 where temperatures are less than 50 degrees Celsius.

Preservative treatment, to Hazard H5 of AS 1604, may enhance this protection but the choice is important to inhibit chemical leaching of the preservative out of the pile by ground water.

If such ground water is acidic, the preservative employed should be an oil-based type, e.g. pigment emulsified creosote. In alkaline ground water the preservative employed should be a multi-salt, fixed waterborne type, e.g. copper chrome arsenic.

Where treated timber piling is exposed to tidal salt water, e.g., wharf piling, the timber should be impregnated with preservative to resist the likely attack of marine boring organisms, unless this hazard has a ‘low’ rating, e.g. cool southern waters. A dual treatment should be applied as two separate impregnations, firstly with copper chrome arsenic and secondly with pigment emulsified creosote to Hazard H6 of AS 1604.

4.4 REFERENCES

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SECTION 5 TESTING

5.1 INTRODUCTION

5.1.1 Preamble A variety of pile test methods from which an appropriate test is selected to provide the data required is available as follows:

(a) Static pile load test, to measure pile performance under full scale loading.

(b) Dynamic pile load test, to predict performance by analysis of dynamic impact loadings.

(c) High or low strain pile integrity test, to ascertain the structural reliability of a pile by examination of reflected stress waves from blows by pile hammers and hand held hammers.

(d) Alternative testing methods.

A pile test is usually performed for the purpose of—

(i) finding the pile movements to be expected under applied loading conditions;

(ii) assessing the ultimate strength;

(iii) assessing the load distribution down the length of the pile or checking the structural soundness of the pile;

(iv) assessing the stiffness of the pile-soil system, from which the soil modulus may be assessed and hence enabling the movement of pile groups to be predicted with increased confidence; and

(v) assessing the integrity of the pile shaft.

It is appropriate to precede specifications for pile testing by some general comments regarding the reasons for and the applicability of testing and the interpretation of results.

5.1.2 Types of test program

5.1.2.1 *General* Piles are usually tested using direct measurements by applying a load (compression, tension or lateral) and measuring the resulting pile movement. In some instances, instrumentation of the pile shaft is added to provide greater information from the tests. Depending upon the size and type of foundation, pile load tests may be performed at different stages of design or construction.

The types of testing program may be summarized as follows:

(a) Pre-contract test programs.

(b) Preliminary test programs.

(c) Routine proof testing programs.

Test programs may comprise load testing by either static or dynamic methods and may include integrity testing, or a combination of these methods.

5.1.2.2 *Pre-contract test programs* These programs are often conducted to confirm design assumptions. The following points should be considered to maximize the benefits of testing—

(a) a detailed site investigation should be carried out at the test location;

(b) the piles and installation equipment and construction method should be the same as those intended for the construction of contract piles;

(c) the pile installation should be observed and documented in detail;

(d) the piles should be loaded to failure wherever possible; and

(e) the provision of instrumentation to measure the transfer of load from the shaft and the toe of the pile to the soil, may provide additional information.

Pre-contract testing is particularly appropriate on larger or technically difficult projects. The pre-contract tests allow refinement of design assumptions and allows optimization of pile type, size and load capacity.

5.1.2.3 *Preliminary test programs* Preliminary test pile programs carried out at the beginning of the piling contract provide much the same information as pre-contract programs, but also indicate, within a tolerance, the performance of contract piles without the disturbance effects of adjacent piles. Further piles installed during a contract should perform, within a given tolerance, in a similar manner. Non-working preliminary piles may be tested to failure, particularly in the absence of pre-contract piles, in order to assess the ultimate geotechnical strength.

This type of program has the benefit that test pile installation is definitely achieved using similar equipment and methods proposed for the contract piles, a situation intended but not always achieved using pre-contract test programs.

5.1.2.4 *Routine proof testing programs* Routine proof testing involves testing of a representative number of piles during the contract, usually in compliance with quality control requirements. These tests may be used to—

(a) check that the specified performance criteria are being achieved;

(b) check the validity of design assumptions;

(c) check that the method of installation results in the production of structurally sound piles for the expected range of soil conditions.

Static load testing will ordinarily only detect major structural deficiencies, whereas high and low strain integrity testing may detect relatively smaller deficiencies. In the case of driven piles, check that pile driving equipment maintains consistent performance, as expected or specified, during the course of the contract, by dynamic monitoring.

The extent of the test program will depend upon the confidence applicable to any one site. This confidence may vary according to previous experience in the area (particularly for the same pile type) and variability or knowledge of the soil conditions, the construction methods proposed and the capital cost of the works. It is therefore difficult to be specific regarding the appropriate number of piles to be tested. On small projects, the cost of testing may be prohibitive and the professional engineer must rely on his/her own or the piling contractor’s experience and expertise. On larger projects, testing requirements will increase and may justify a substantial number of tests, possibly including extensively instrumented piles.

The selection of the piles to be tested is usually made on the basis of observed installation behaviour or documented installation records, selected piles generally being those with anticipated ultimate geotechnical or structural strength.

It is not unusual to combine static load testing with other test methods (e.g. dynamic), to obtain representative testing for the project.

5.1.3 Selection of pile load The selection of the maximum pile load to be applied is dependent on the purpose of the test.

As a general rule, precontract and preliminary test programs should ideally establish the ultimate strength of the piles.

In cases where static load tests are performed on contract piles, the recommended procedure is for a maximum loading equal to the design strength load.

In the case of dynamic testing, the ultimate geotechnical strength is often mobilized. An exception to this is with piles under conditions of effective refusal to driving when the full geotechnical strength of the pile cannot be mobilized and the test reverts to a proof load test.

For tension testing of piles which are also to be utilized for compression loading, there may be a need to limit the maximum pile movement, or to provide for redriving of piles after testing.

AS 2159, Clause 8.3.4 describes static pile testing procedures for compression loading. These are intended to provide a basic framework which sets out the general principles for good quality practice. They are applicable primarily to proof load testing and it is recognized that for more complex testing (e.g. for research purposes and precontract testing to assess ultimate load performance), the procedures may need to be modified to suit particular circumstances.

As indicated, the loading time cycle is designed for most of the measurement to be completed within one day, with possibly one overnight cycle. More accurate assessment, say of creep performance, may require longer cycle and increment times.

For tests which are taken to assess ultimate strength, the guidelines of the load schedule (see AS 2159 Figure 8.1), may be utilized, but consideration should also be given to the use of constant increments of deflection rather than load, to obtain a better definition of the load-deflection performance.

The loading procedures for all static load testing in AS 2159 are intended as ‘default’ procedures which may be modified by the designer to suit particular circumstances. Where modifications are made however, test reports should clearly state the departures from AS 2159 requirements and state the reasons for such departures.

5.1.4 Pile acceptability For most situations where piles are tested, it is required that pile performance meet some specified criteria for pile head movement under application of load. The criteria for acceptance should be considered by the designer, with regard to the following—

(a) required ultimate geotechnical and structural strength;

(b) tolerance of the structure to settlement—both total and differential;

(c) group action of piles; and

(d) the pile head movement characteristics associated with ground conditions and the elastic behaviour of the pile performance. The professional engineer must ensure that these movements will be compatible with the structure.

AS 2159, Section 8.3 provides default requirements in which the following rationale has been adopted:

(i) The requirements relate primarily to piles for conventional building construction, with strength and serviceability loads broadly in the range of 300–3000 Kn, founded in soil or on rock. More heavily loaded piles which are generally associated with multistorey construction may have more stringent requirements for performance.

(ii) Adequate settlement performance and adequate ultimate geotechnical and structural strength are the most important features.

(iii) Long or slender piles may experience significant elastic compression of the pile. This would be reflected in the test pile performance. The professional engineer must ensure that these elastic movements will be compatible with the structure.

5.2 STATIC COMPRESSION LOAD TESTING OF PILES

5.2.1 Introduction Static load testing is the measurement of pile settlement response to the application of time related pattern of full scale loading. Thus the actual load applied and the deflections recorded provide a full scale comparison with the loads and deflections in practice. This is the significant area of differentiation between this and dynamic testing. The results obtained are actual observed values and not estimated values.

5.2.2 Delay between installation and testing The ultimate geotechnical strength of displacement piles founded in clays generally increases with time. This is predominantly due to dissipation of pore pressures, but soil and water chemistry may also contribute to the increases in strength. Both the magnitude and rate of the strength gain depend upon the ground conditions, pile type and the method of pile installation. In certain sedimentary rocks, pile relaxation (loss of geotechnical strength with time), has been documented.

Load testing of piles should not commence until the strengths of materials in the pile and pile cap are adequate to sustain the maximum test load.

Piles in sand or end-bearing piles on rock may be tested upon completion of installation but friction piles in cohesive soil, or end bearing piles in dense silty sands may require longer periods. With some clay soils, substantial strength increases may continue for several months and while delay periods of longer than four weeks are uncommon in practice, they may need to be considered. Dynamic testing may provide a cost effective means of estimating the strength increase with time (see Clause 5.5).

5.2.3 Acceptance criteria The acceptance criteria defined in AS 2159, Clause 8.3.5.6 and Table 8.2 are recommended for normal construction, i.e. individual piles or pile groups at minimum spacings of several metres supporting structures that do not have total or differential movement constraints more stringent than usual. The applicability to the project of the acceptance criteria given in AS 2159 should be considered during design. Where a project requires different acceptance criteria, the test pile acceptance criteria should be defined in the Schedule of Load Test Requirements (see AS 2159, Figure 8.1).

5.2.4 Constant rate of penetration test (CRP) The CRP is an alternative to the quick maintained load (QML), test described in AS 2159, Clause 8.3.4. In the CRP test, the pile is made to penetrate the soil at a constant rate while the force at the head of the pile is continuously monitored.

When a CRP test is required, the rate of penetration should be in accordance with Table 5.1.

TABLE 5.1

RATE OF PENETRATION

Soil type	Rate of penetration mm/min
Clay	0.5
Sand and gravel	2.0

NOTES:

1 Loading should be continued until a constant load or penetration of at least 10% of the pile diameter has been achieved.

2 Tests have shown that the actual rate of penetration, provided it is steady, may be half to twice the above values without significantly affecting the result.

3 It is advisable to specify that a plot of load against penetration is to be made during the course of the test, so that the ultimate bearing capacity of the pile may be readily identified, enabling termination of the test after an appropriate penetration has been achieved.

5.3 TENSION (UPLIFT) LOAD TESTING OF PILES Tension loads on piles are often intermittent or cyclic rather than permanent. Selection of a load schedule approximating to the loading condition of the pile in service would be expected to provide the most useful data. Variations from an incremental sustained load test as detailed in AS 2159, Clause 8.3.5 include the following:

(a) Applying cyclical load.

(b) Loading at constant time intervals.

(c) Constant rate of tension method.

(d) Quick load test method.

Additional information on these alternative loading arrangements and other details of tension load test procedures may be obtained from ASTM D 3689.

Where piles will be subjected to both compression and tension loads, a load schedule involving compression and tension loads may be appropriate.

5.4 LATERAL LOAD TESTING OF PILES Lateral loads on piles are often intermittent or cyclical rather than permanent. Selection of load schedule approximating the loading condition of the pile in service would be expected to provide the most useful data. Variations to the loading schedule in AS 2159 include the following:

(a) Applying cyclical loading.

(b) Loading to a specified total lateral movement.

(c) Reverse loading (test pulling in one direction, then repeat test pulling in opposite direction).

(d) Reciprocal testing (apply each load movement first in one direction, then the other).

Additional information on these alternative loading schedules and other details of load test procedures may be obtained from ASTM D 3966.

5.5 DYNAMIC PILE TESTING

5.5.1 Introduction Dynamic pile testing is based on the measurement of compressive stress-waves generated in a pile from an impact at the pile head, and the reflected stress-waves from the pile and soil or rock. The magnitudes of the reflected stress-waves are a function of the amount of soil resistance and the timing of the reflections allows the distribution of that resistance to be estimated. The stress-wave in a dynamic pile test is usually generated by a piling hammer.

This test was developed initially for preformed and driven piles, such as reinforced or prestressed concrete, steel and timber piles. However, the test method has been extended to cast-in-place piles.

Dynamic pile testing is normally used to estimate the pile capacity, soil resistance distribution, immediate settlement characteristics, hammer transfer efficiency and pile stresses during driving. In addition, the location and severity of damage to preformed piles or construction irregularities in case-in-place piles may be assessed.

It should be noted that the accuracy of prediction of resistance distribution along the pile is less precise than the load-settlement prediction at load levels below the peak load mobilized by the dynamic test. This is particularly so for large diameter, long, bored piles socketed into rock and where the shaft cross-section varies appreciably.

Insufficient correlative information is currently available from instrumented static pile load tests for this type of pile to quantify the accuracy of the dynamic method prediction regarding the resistance distribution.

The test results can be applied to:

(a) Assess or verify pile design at the precontract stage or at subsequent stages of the contract.

(b) Proof-test select contract piles.

(c) Estimate pile stresses in the pile shaft during installation (to avoid pile damage).

(d) Detect or confirm pile damage.

(e) Assess hammer energies to check input for pile driving formulae.

(f) Estimate parameters for input to wave equation analyses.

(g) Check assumptions made of pile driveability.

5.5.2 The test method Test piles are instrumented with transducers, to measure transient strain (or force) and acceleration (or velocity), at a distance of preferably two-pile diameters or more from the pile head. The test data for each hammer impact, or from selected hammer impacts, is recorded in either analog or digital form for further analysis and subsequent storage.

It is fundamental to dynamic pile-testing that the soil resistance to pile motion is considered to comprise static, dynamic and inertial components. However, alternative commercial or research systems may differ in detail about the models used for these components. Closed-form solutions, such as the CASE method, have been devised to estimate the static component of driving resistance during testing, but they may not be accurate as they rely on a suitable choice of damping factor.

The results thus obtained, although useful in providing a quick guide to the static pile capacity, can at times be quite inaccurate without suitable monitors such as a static pile load test or other factual load test data.

To this point the testing and analysis (closed-form analysis and digital read out), has been done on site with immediate answers. To verify or correct these results, the next step is to do more detailed analysis, based upon wave-equation philosophy. This is normally done remote from the site and is run by computer program as a process of signal matching (e.g. CAPWAP and TNOWAVE). Besides providing a better prediction of static pile capacity these programs can also estimate the pile bearing resistance distribution and the immediate pile movement characteristics under static load.

Dynamic pile testing and particularly the analysis of test data, must be supervised by appropriately trained personnel.

It is important to note that since the dynamic test load is applied for a very short duration, inferred settlement under static load does not include any long term effects such as consolidation or creep. Thus, although dynamic pile-testing will provide some insight into pile-soil interaction, long term settlements will not be predicted and, if these are of concern, appropriate geotechnical methods of calculation should be used, or one or more static load tests should be performed to better estimate these settlements. Creep settlements at typical long-term structural load levels, are commonly not a major consideration.

Estimation of the ultimate geotechnical strength may be obtained by increasing as necessary the hammer energy applied to the pile, provided that this can be achieved without damaging the pile. A notable exception to this is the situation where the end bearing strata is relatively strong and the majority of geotechnical strength is obtained through end bearing. Application of a hammer blow to these end bearing piles may not result in sufficient pile movement to mobilize the ultimate geotechnical strength. In this event, the test reverts effectively to a proof load check, with the proof load determined by the applied hammer energy. In such testing, care should be taken not to damage the pile by overdriving.

Where the minimum load carrying capacity of piles cannot be satisfactorily verified by dynamic testing without the risk of damage, static load testing may be required. This is particularly relevant in the case of large diameter, deep-bored piles, where limited experience suggests that the ultimate geotechnical capacity of piles may be substantially underestimated by dynamic testing.

The time that should elapse before piles are dynamically tested needs to be considered carefully. Ideally, this should have regard to soil characteristics and pile installation processes, but practical and economic factors will influence decisions and test information requirements. Where it is required to estimate pile driving stresses, testing should be carried out during the installation process. Tests performed during installation may also provide the best estimate of end bearing, where substantial post-installation increases in shaft resistance (‘set-up’), may reduce the energy available to mobilize end resistance after installation. Conversely, tests performed some time after installation (‘restrike tests’), will provide the best estimate of long-term shaft resistance and of total pile capacity if sufficient movement can be generated at the pile toe. In certain sedimentary rocks, particularly for close-spaced pile groups, pile relaxation (loss of strength with time), has been documented.

5.6 ALTERNATIVE TESTING METHODS In addition to static and dynamic load testing, so-called pseudo-static test methods are becoming available. These tests are a form of dynamic test, in which the loading rates and induced pile velocities are slower than for the normal impact tests. The principal advantages of such tests are minimized dynamic effects and lower stress levels in the piles. These methods are still in their development stages, but may be used given appropriate validation and technical justification.

5.7 INTEGRITY TESTING

5.7.1 Introduction A number of methods are available to check piles to assess whether the structural integrity of a pile is satisfactory and hence capable of performing its design function. These include—

(a) excavation;

(b) load testing;

(c) diamond coring to retrieve concrete samples (may be unreliable for checking pile/rock interface);

(d) probe methods, using sonic or nuclear probes inserted into preformed tubes in the pile to obtain a measure of transmission or neutron absorption at various levels;

(e) low-strain methods; and

(f) high-strain methods.

The most common methods presently used are the low strain and high-strain methods. High-strain methods are described in Clause 5.5 while Clause 5.7.2 is restricted to low-strain methods.

5.7.2 Low-strain integrity testing

5.7.2.1 *General* Development of low-strain methods has not reached a stage where it is appropriate to propose standard methods for testing and interpretation. Therefore this Section is limited to a brief description of available tests and some broad guidelines on their application and usage.

There are various types of low-strain tests, including sonic tests, frequency response tests, echo tests and similar. The main low-strain methods used in Australia at present are:

(a) *Sonic method* In this method, an impact is imparted to the head of a pile by a light mallet and the response measured by a hand-held transducer. Discontinuities and changes in pile cross-section produce reflections, which change the shape of the signal. The signal is interpreted according to its shape. Major defects, together with an indication of the depth at which they occur, should be detected.

(b) *Frequency response method* This method also requires an impact to the head of a pile by a light mallet. The frequencies of the response are monitored, producing resonance peaks which are used to display major defects. The stiffness of the pile head is also able to be determined with this method.

In both methods, the signals are not only affected by the pile geometry and discontinuities, but also by soil characteristics which can greatly damp the signals. Hence, interpretation of the signals should be made only by trained and experienced personnel, who must be provided with all available information on pile construction and installation, together with a knowledge of the soil conditions believed to be present for the piles tested.

A skilled operator is usually able to judge during the performance of the test whether a pile may be regarded as structurally reliable.

Many low-strain methods also have associated software which permits a prediction of the actual pile shape to be made.

Unlike many other forms of integrity testing, low-strain testing is a non-destructive test which requires no special preparation during or after construction of the pile, apart from providing a sound pile head for applying the hammer blow.

Concrete, grout-injected, timber and steel piles may be tested satisfactorily, although the overwhelming number of tests are performed on concrete and grout-injected piles.

5.7.2.2 *Test requirements* It is fundamental that the highest quality signals be obtained for interpretation and this is more likely to be achieved by observing the following:

(a) Concrete or grout piles should be trimmed back to sound material, free of loose surfaces, cracks and debris.

(b) The surface should be free of water.

(c) Any structures or elements attached to the pile, e.g. long projecting reinforcement or cages, may return signals generated by these elements which may make the signal impossible to interpret. Often, interference from these elements may be electronically filtered out. However, where possible, the head of the pile should be kept clear until testing has been completed.

(d) Access to the head of the pile should permit delivery of a number of hammer blows and provide sufficient room for movement of the transducers around the pile head. Testing with heavy pile cap reinforcement cages in place may be difficult or impossible to perform.

(e) ‘Green’ concrete should not be tested. Cast in place piles normally require a minimum curing time of 3 to 7 d prior to being ready for testing.

(f) A number of blows should be delivered to each test pile to ensure repeatability and hence consistency of results.

5.7.2.3 *Advantages of low-strain testing* These include the following:

(a) The equipment is fully portable and a single operator may be able to test up to 100 piles per day or more, subject to site conditions and access.

(b) The testing results in minimal interference with construction activity.

(c) Defects may be discovered at an early stage.

5.7.2.4 *Limitations of low-strain testing* These include the following:

(a) The method is usually not suitable for mechanically jointed piles (which should then be tested using high-strain methods).

(b) Piles already cast into a large pile cap are generally not suitable for testing.

(c) A toe reflection is not always obtained. End-bearing piles with low frictional damping usually give a toe reflection if the length is less than 60 pile diameters; piles having large frictional damping usually show a toe reflection when the pile length is less than 30 pile diameters.

NOTE: Detection of the pile toe alone is neither an indication of the deficiencies of the test equipment, nor a means of establishing the validity of the test. For a pile having a slenderness ratio well below the limits suggested above, founded in hard rock, it may not be possible to detect the toe using low or high strain methods. However, comments relating to shaft integrity may still be made).

(d) Minor cross section deficiencies (less than 5%), cannot be detected.

(e) A continuous crack across the completed cross-section will appear as the pile toe, even if the crack is of hairline thickness only. This may not be of consequence for a pile subjected to compression loading only.

5.7.2.5 *Phenomena usually detected* Low-strain tests will usually detect the following phenomena or combination thereof—

(a) reflection from the toe;

(b) reflection from any significant inclusions of material having different acoustic properties;

(c) reflections from a crack normal to the axis of the pile;

(d) reflections from a joint (e.g. as in a precast pile);

(e) reflections from an increase in cross section;

(f) reflections from a decrease in cross section;

(g) reflections from changes in soil properties; and

(h) reflections from material changes.

5.7.2.6 *Phenomena usually not detected* In general, the following will not be detected—

(a) a gradual increase in cross section;

(b) a gradual decrease in cross section;

(c) a curved form;

(d) small inclusions of foreign materials;

(e) local loss of cover;

(f) debris at the toe of the pile; and

(g) a crack parallel to the pile centre-line.

5.7.2.7 *Summary of low-strain testing* Includes the following:

(a) Low-strain integrity testing provides a quick appraisal of the pile in place.

(b) The tests are incapable of demonstrating the ultimate geotechnical or structural strength of the pile.

(c) The tests should not be used as a final arbiter of good or defective piles, but as an initial tool to detect possible major defects. Generally, consistency of signal characteristics are often the first guide to determination of any significant anomalies. A poor integrity test result warrants additional investigative work.

(d) Natural rules of physics determine the limitations of the test.

(e) Pile depth determination is not always possible.

(f) Small or gradual changes of soil conditions or pile section cannot be detected.

(g) In some ground conditions the test cannot distinguish between a reduced diameter of pile (neck) and an increased diameter changing to the normal diameter.

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