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DESIGN STRENGTH OF VARIOUS HOUSE PILE FOUNDATION SYSTEMS

S. J. Thurston

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PREFACE

This report describes the first phase of a research programme undertaken by BRANZ, which evaluates the strength of house pile foundations when subjected to simulated earthquake motion. The second phase will address problems identified in phase 1 and, in addition, develop a method by which the performance of foundations subjected to lateral loading may be evaluated directly from the laboratory test data.

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NOTE

This report is intended for structural engineers, architects, designers, manufacturers, code writers, researchers and others involved in the field of house foundation study.

The work reported here includes the assessment of some named proprietary systems. See Appendix D for details.

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S J Thurston

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Piles; Foundations; Houses; Residential buildings; Timber; Design loads; Bracing; Earthquake loads; Wind loads

ABSTRACT

The assumptions and philosophies used to derive design loads for pile foundations in light timber frame buildings are critically examined. Recommendations on changes to the non-specific design code (NZS 3604) are made. Experimental test results for braced and cantilevered piles are provided. A proposed foundation test and evaluation procedure is also presented. Areas needing additional work are identified.

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1.0 INTRODUCTION

New Zealand houses built this century have generally performed well under severe wind or earthquake loading. Earthquake damage has been limited to collapse of unreinforced brick chimneys (and to a lesser extent brick veneers), failure of some forms of pile foundations and, in a few instances, racking failure of lower storey walls (Cooney, 1979). Wind damage is usually limited to failure of roof systems (sheeting and framing). Foundation failures have usually been attributed either to inadequate joint detailing or to the failure to provide a load path through which lateral loads can be transmitted from the structure to the ground. These issues were partially addressed in SANZ (1978). Engineering principles have been more rigorously applied in SANZ (1990a). However, as no major earthquake has occurred since the 1931 Napier quake, modern construction has not been well tested.

The assumptions and philosophy used to derive the design loads for the Standard for Light Timber Frame Buildings not requiring Specific Design - NZS 3604:1990 (SANZ, 1990a) - have not been well publicised. The 1990 Standard developed an engineering rationale for the complete design process, from derivation of design loads through to provision of load resisting elements with a traceable load path from the loaded element down to the foundations. It has long been acknowledged, however, that timber framed buildings have greater resilience than expected by application of strict engineering principles. Such reserve strength comes from their system behaviour which ensures that load sharing and composite action occurs under wind or earthquake loading. This enhances overall building performance. In addition, houses have many "non-structural" elements which contribute significantly to their strength. Designs which are structurally conservative are generated using the Standard because it covers a large variety of forms of construction.

This report examines some of the assumptions applied to the foundation and sub-floor section of the Standard. It compares and critically discusses these assumptions using published data and the experimentally derived results presented in this report. Recommendations are provided where anomalies were uncovered. Areas where more research is required are highlighted. The need for a foundation test and evaluation procedure is demonstrated and a solution proposed.

2.0 BACKGROUND TO NZS 3604:1990 FOUNDATION DESIGN LOADS

Lapish (1991) provided engineering calculations which had been used by the NZS 3604 Standards Committee during preparation of the Standard (SANZ, 1990a). Some of the assumptions implicit in Lapish's calculations are discussed in various places in this report.

The Standard stipulates the level of earthquake and wind forces that must be resisted. The earthquake forces determined in SANZ (1990a) for foundations were for a category IV building with a natural period of vibration (T) of 0.4 seconds, and a structural ductility factor (μ) of 4. SANZ (1990a) uses a simplified form of the zone factor from SANZ (1992a). The wind loading in SANZ (1990a) was also derived from the Loadings Standard but is presented in a simplified form (Thurston and King, 1992).

NZS 3604:1990 (SANZ, 1990a) stipulates ultimate limit state design strengths for many structural elements. The writer ensured that design strengths assigned to elements were arrived at using the same assumptions used in deriving building loads. For instance, a structural ductility (μ) of 4 was assumed when deriving earthquake loads. If a seismic resisting element was not capable of sustaining deformations to this ductility, then the assigned strength was appropriately downgraded.

Interestingly, the period T was taken as 0.6 seconds (rather than 0.4 seconds) when deriving loading on bracing walls (Lapish, 1991). This results in reductions of over 20% in earthquake design loads.

Direct use of engineering rationale, ignoring strength enhancing features that are difficult to quantify, will result in a very conservative structure. There was a conscious attempt by the NZS 3604 Standards Committee to avoid elevating the strength requirements in NZS 3604:1990, unless there was evidence of inadequate performance in practice. This is reflected in some assumptions made by the writers.

2.1 Design Loads for Pile Foundations

2.1.1 Braced Piles

The earthquake design loads specified in NZS 3604 (SANZ 1990a) for braced piles were based on capacity values of bolted timber braces and calculations by Lapish (1991). This is summarised in Equation 1. The forces are given in bracing units (BU), where 20 BU is defined as equal to 1 kilo Newton (kN). The NZS 3604 Committee assessed the ultimate axial strength of a brace as 17 kN from various test results including Wood et al. (1976). The effective horizontal capacity (12 kN) was obtained by vector resolution with the brace at the maximum permitted inclination of 45° (i.e., by multiplying 17 kN by SIN 45°). A reduction factor of $F1 = 0.5$ (based on Dean, 1987) was used to downgrade the "pinched" (S-shaped) hysteresis loops expected for braced piles to equivalent elastoplastic loops. (Note, NZS 4203 (SANZ, 1992a) design loads were derived assuming elastoplastic loops.) Finally, the braced pile was considered capable of sustaining only limited ductility ($\mu = 2$) deformations (as recommended by Dean et al., 1989, for bolts with diameters less than 18 mm) (cf. $\mu = 4$ assumed by SANZ, 1990a). To accommodate this lower ductility a factor $F2 (= 0.27/0.49)$ was introduced. This is the ratio of NZS 4203 (SANZ, 1992a) loadings for $\mu = 2$ and $\mu = 4$ systems at $T = 0.4$ seconds. Thus, Lapish obtained the following design value for braced piles from:

$$12 \times F1 \times F2 \times 20 = 12 \times 0.5 \times 0.27/0.49 \times 20 = 66 \text{ BU} \dots (1)$$

The design value of 66 BU was rounded up to 70 BU for simplicity in NZS 3604. Whereas earthquake loading is cyclic in nature, requiring ductile behaviour under repeated cyclic loading, wind forces are predominantly unidirectional, with peak loading being the coincidence of mean wind plus turbulence. The ductility of the system is relatively unimportant in such instances. For the wind bracing situation $F1 = F2 = 1$, a value of 240 BU is obtained by calculation. The Standard assigns a bracing rating of 160 BU. This includes allowance from other selected results by Wood et al. (1976).

If a building is more flexible (i.e., period T more than 0.4 seconds) or more ductile (μ more than 4) than assumed by SANZ (1990a), the earthquake forces derived by using the equivalent static analysis procedures detailed in the Loadings Standard can be reduced. As the earthquake loads in SANZ (1990a) assume the building has a period $T = 0.4$ seconds and ductility $\mu = 4$, in order to obtain design strengths that are compatible with the Standard, the measured earthquake resistance of elements at different μ and T values must be adjusted. This is done by using Equation 1.

2.1.2 Deep Cantilevered Piles

Lapish (1991) calculated design values for 140 mm diameter driven piles, based on the calculated flexural strength of timber at ground surface level. The failure stress was assumed to be (12.4 x 1.5) MPa. If piles are assumed to be loaded at 1.2 m above the ground (the maximum allowed by NZS 3604: 1990), a wind design load of 4.17 kN results, although the value specified in the Standard is 3.5 kN. Lapish considered that a ductility factor of 1 was appropriate for timber flexural failure. He therefore obtained the earthquake design

load by factoring the calculated strength of 4.17 kN by the ratio of NZS 4203 (SANZ, 1992a) loadings at $\mu = 1$ and $\mu = 4$; thus obtaining 1.56 kN. (1.5 kN is used in the Standard.) A shaving factor of 0.7 and a steaming factor of 0.85 were considered but not eventually used. Tests conducted by Cocks et al. (1975), discussed in Section 2.3.2, verify that soil complying with minimum values specified in NZS 3604:1990 can sustain the above design loads.

2.1.3 Shallow Cantilevered Piles

Lapish (1991) based the design loads for shallow cantilevered piles on limited tests, in which the ultimate load was approximately 2.2 kN when the load was applied 400 mm above the ground. Lapish adjusted this load for a pile height of 600 mm, hence obtaining 1.5 kN - the design wind strength in SANZ (1990a). For earthquake load this was further factored by the F2 factor in Equation 1 (0.28/0.8) to make the results applicable to elastic design (ductility factor of 1). Being elastic design there was no correction for the shape of the hysteresis loops (as was done in Equation 1). After rounding, the design value in SANZ (1990a) became 0.6 kN for earthquake loading. No strength reduction due to earthquake cyclic loading has been taken into account.

Several approximate methods are available for calculating ultimate strength resistance of short piles or posts within soils when limited by soil strength. Some of these methods are discussed in Section 2.4. An estimate of soil lateral bearing strength is required. Lapish's (1991) calculations for pile capacity using the Rutledge formula (Patterson, 1969) assumed 100 kPa soil lateral bearing strength. This resulted in a strength of 2.9 kN for a pile loaded at 600 mm above the ground. However, this calculation assumed a pile footing diameter of 350 mm, which exceeds the 230 mm minimum allowed by NZS 3604 (SANZ 1990a). This design force is significantly higher than the 1.5 kN load Lapish obtained from the test results discussed in the paragraph above.

2.1.4 Anchor Piles

SANZ (1990a) requires 12 kN connections between joists and bearers and between bearers and piles to transmit lateral load from floor to pile. (This load was adopted to match the assumed horizontal strength of a brace.) Based on this 12 kN force, Lapish (1991) used Equation 1 to derive the design earthquake force for anchor piles. He also adopted the braced pile wind capacity. Calculations in Section 2.4 look at maximum anchor pile capacity based on the soil strength and pile flexural strength respectively.

2.2 Braced Pile Design Loads from NZS 3603

The Standard for Timber Design, NZS 3603 (SANZ, 1990b), provides a (working stress) design strength for 12 mm bolts in single shear (when subjected to earthquakes and loaded at an angle of 45° to the grain of No. 1 Framing Grade Radiata Pine) of $1.52 \times 1.75 = 2.65$ kN. Walford (1992) suggests that this should be increased by 69% (being a ratio of 1.35 ultimate/working load divided by a 0.8 capacity reduction factor) to obtain the ultimate design strength (giving 4.48 kN). Parallel to the grain, the ultimate design value is $2.89 \times 1.75 \times 1.69 = 8.55$ kN. These ultimate load design strengths are lower than the ultimate brace force of 17 kN at 45° to the grain used by Lapish (1991) or the experimental test results discussed in Section 2.3.1. For normal brace dimensions, design values in NZS 3603 suggest brace capacities will be determined by bolt design loads rather than brace buckling strength.

2.3 Previous Work

2.3.1 Bolt Tests at BRANZ and WORKS Central Laboratories

Wood et al. (1976) cyclically tested a small number of 12 mm bolted joint connections in single shear between small assemblies of timber braces inclined at 45° to joists. The lowest load was 17.6 kN and the average, 24 kN. Cyclic degradation and its influence on ultimate load was small. Most failures were by joist splitting which is influenced by joist flexural stresses. Thus, the results cannot be applied directly to more typical (longer) joist spans. Bolt slip exceeded 20 mm at failure in all cases. Other (limited) tests by Wood et al. on braces indicated that brace buckling would not be a problem with typical New Zealand house construction.

Spurr and Phillips (1984) tested single bolt timber joints loaded both parallel and perpendicular to the grain. Joint slips at failure generally exceeded 20 mm, and the measured forces were well in excess of the Timber Standard design strength. The average maximum load and deflections at failure for specimens loaded perpendicular to the grain were approximately 80% of the corresponding values for specimens loaded parallel to the grain. The lower 5 percentile strength of a 12 mm bolt loaded parallel to the grain was 15.6 kN.

Harding and Fowkes (1984) and Gerlich (1988) investigated the effects of departures in end and edge distances, larger bolt hole, and reduced washer size from the Timber Design Standard (1990b) requirements. The former two variations had a significant effect on joint ultimate load; the latter two had little effect.

Thurston and Jacks (1985) performed monotonic tension tests on bolted joints using minimum end and edge distances and 12 mm bolts to ascertain design loads for various bolting configurations. One, two, and four bolt joints, bolts staggered and aligned were tested in single and double shear. The lower 5 percentile strength of a single 12 mm bolt loaded parallel to the grain was 18.1 kN and the average value was 27 kN.

2.3.2 Round, Driven Deep Cantilevered Piles

Cocks et al. (1975) conducted a series of tests on 1.8 m long 140 mm diameter timber piles driven 1.2 m deep into Auckland soil. These were laterally loaded at the top. There were 10 sites and the soil was generally classified as firm/stiff silty clay. Notably absent were sand or gravel sites. Three sites had Scala Penetrometer soundings of between 3 and 4 blows per 75 mm penetration, three were between 2 and 3, and three between 1 and 2. (One site was unknown.) These are near to or below the minimum soil strength specified in NZS 3604 (SANZ, 1990a). However, the average loads/pile at 20 mm deflection at the pile top for the above soil groups were 8.2 kN, 6.9 kN and 5.4 kN, respectively (excluding one site where the results appeared excessively high). These resisted forces are well in excess of the ultimate limit state loading to be applied as specified in NZS 3604: 1990 although the results are not applicable to non-cohesive soil. The pile deflections at ground level were generally slightly more than half that at the pile top.

2.4 Pile Lateral Load Capacity in Soil

To be of general application, foundation capacity must be based on the weakest soil conditions consistent with the scope of application of the intended use (namely, clause 3.1.1(a) of NZS 3604:1990). This clause requires the soil to have a safe (vertical) bearing strength of 100 kPa below the underside of the pile. This can be ascertained by either general observation of the performance of nearby buildings, or by performing Scala Penetrometer soundings at depths below the pile founding depth. The maximum penetration allowed from this testing is 25 mm/blow. For this Scala Penetrometer result, Stockwell (1977) suggests that an appropriate allowable bearing pressure is 125 kPa (using a safety factor of 3), and the soil is likely to be either

stiff clay, uniform compact sand or well graded loose sand. Note, SANZ (1990a) places no limitation on soil strength over the depth of the pile. Yet it is this upper soil zone which determines pile lateral resistance. For this report, soil in this zone is assumed to be soft to medium clay. The actual strength will vary extensively from season to season (due to moisture content changes) as well as place to place throughout New Zealand. Terzaghi and Peck (1967) suggest that the unconfined compressive strength of soft clay varies between 25 and 50 kPa; values for medium clay are 50 to 100 kPa. On this basis, an unconfined compressive strength of 50 kPa has been used in this report for soft to medium clay. If the piles pass through a layer of peat or very soft clay before being founded in a firmer soil, then the lateral pile stiffness will be lower. The spacing of piles used in normal house construction means "group" effects will usually be negligible.

In general, free standing house piles act as cantilevers and must resist both a moment and a shear force, as shown in Figure 1. However, in some situations the pile is effectively rotationally restrained at the top, in which case the pile embedment depth need only be designed to ensure the soil resists the applied shear load. A braced pile, or pile fastened to sheet bracing, approaches this horizontal shear transfer situation. This situation is discussed further in Section 2.6.5.

Although long pile design is usually governed by deflection (WORKS, 1990), short piles are often limited by soil strength. If house piles deform excessively in the ground during an earthquake, there is a danger that building services may rupture and P-Delta effects may become significant. However, these large deflections mean that pile deformations will induce some form of "base isolation" and earthquake forces may be reduced from those assumed for the rigid foundation example. This is discussed further in Section 2.7.

It is essential that a suitable ultimate limit state deflection criteria for foundations be determined. A suitable limit may be that ground floor deflection does not exceed 60 mm under the NZS 3604:1990 (strength) design load. The soil should be taken as possessing the minimum strength allowed by the Standard in this analysis and the effects of cyclic load considered. For an anchor pile embedded 900 mm deep into the soil, if the rectangular soil model outlined in Appendix B is assumed, then the corresponding pile deflection at the ground level is approximately half that at the pile top. Experimental testing (Cocks et al., 1975) obtained a similar result.

Serviceability criteria for pile foundation design will also need to be considered, to ensure excessive deflections or vibrations do not occur at serviceability loads. Determination of suitable criteria is recommended as a separate study. The relationship between building foundation deflection and onset of damage to building services is also recommended as a study project. It is suggested that a certain degree of damage to building services is acceptable during major earthquakes although only minimal or no damage should occur during minor (serviceability level) earthquakes.

Poulos and Davis (1980) provide formulae for calculating rigid pile deflections as a function of soil modulus of subgrade reaction, either assumed to be constant or to vary linearly with depth although it is questionable whether the formulae are applicable to the squat piles studied in this report. These are reproduced in Appendix B and deflections for anchor and short cantilevered piles are calculated, assuming recommended values of soil parameters. No account is taken of cyclic loading. These predictions vary largely, suggesting that experimental testing of these two types of piles in "minimum NZS 3604 soils" is required if realistic values are desired. Generally, the predictions show that the pile deflections in the soil are acceptable in stiff over-consolidated clays but will govern pile design in soft normally consolidated clays. Deflections (up to failure) will be small in sands not subject to liquefaction.

Poulos and Davis (1980) also stated that in purely cohesive soil, the ultimate lateral resistance increases from $2 C_u$ (undrained shear strength) at the surface to $8-10 C_u$ at a depth of about three pile diameters and remains constant for greater depths. For the squat piles used in house construction, it is reasonable to limit the upper stress block (S1 in Figure 1) to $2 \times C_u$. As the mid-depth of the lower stress block (S2) is at a depth of about 675 mm (1.5 pile diameters) for anchor piles (see Figure B.2), then averaging the above values results in a peak value for S2 of $5 \times C_u$. Equations for calculating short pile lateral loading strength assuming a rectangular soil stress block are presented in Figure B.1. This method is adopted in this report and calculations are made where the allowable (ultimate limit state) lateral pressure (S1) = $2 \times C_u = 2 \times 50 = 100$ kPa. Calculations in Appendix B show that this gives similar wind bracing capacity for shallow cantilevered and anchor piles to those specified in SANZ (1990a). Lower values will be obtained in loose sands.

An alternative short pile design method (based on an elastic parabolic stress distribution) is also given in Appendix B (Figure B.3) and this is also applicable to cohesionless soils. A third method, based on experimental testing in the USA, is the well known Rutledge formula (Patterson, 1969), which is intended to result in 12 mm deflection at the soil surface. This latter method was used by Lapish (1991). Another analytical method is to read the lateral strength directly from charts derived by Broms (1964a, 1964b) which are reproduced in Figure B.2 from WORKS (1990). Broms assumes maximum pressures of $9 C_u$ (unconfined compression strength) in cohesive soils and 3 times the passive pressure over the pile surface in cohesionless soils and uses rigid body rotation to calculate the lateral load. The soil resistance for a depth of $1.5 \times$ pile diameter is ignored which makes the method inappropriate for use for the squat piles typically used in houses. A comparison of the above methods is given in Appendix B. Di Gioria et al. (1981) and Vallabhan and Alikhanlou (1982) developed computer models to predict the behaviour of short rigid piers and obtained good agreement with actual measurements. Both models required input of soil properties to develop equivalent lateral soil "springs". The scope of this project did not allow the models used by these authors to be applied to New Zealand house pile systems. Based on experimental work, Greensill (1990) recommended a simple interaction diagram for ultimate strength predictions for short piers subjected to combined uplift and lateral load. This method was confirmed by subsequent work by Graham (1991). The effect of wind uplift forces was shown to significantly reduce the lateral pile strength and it may be necessary to consider this effect when assigning design pile strengths for new houses in New Zealand.

A literature survey and experimental project to determine the lateral resistance of short piles in soils with the minimum strength complying with values specified in SANZ (1990a), is recommended to provide confidence in the design procedures currently being used. The experimental work would probably involve casting a series of pile "pairs" (ordinary, shallow cantilevered, braced and anchor piles) in concrete within a variety of soils. Reverse cyclic loading would be by jacking one pile against the other and measuring the load deformation characteristics. Some realistic axial gravity load could be added to correctly model friction at the bottom of the pile. Square and round pile footings could be compared and fixed head piles (pile rotation at top restrained by sheet cladding) also tested. Soil measurements would include Scala Penetrometer and soil description. Density and sieve analysis would also be required for granular soils.

2.5 Potential for Pile Flexural Failure

SANZ (1990a) foundation earthquake design loads were derived from the Loadings Standard NZS 4203 (SANZ, 1992a), assuming that the load is limited by "yielding" of the walls or foundations and that the ductility is 4. Where tested elements cannot provide this ductility, the bracing rating evaluation method results in the bracing rating calculated from the test results being downgraded. The assumption is made that no premature brittle failure of components (such as diagonal braces, brace connections or pile fracture) will occur. To ensure brittle failure is avoided, the brittle components should ideally be designed for the lower of the capacity strengths of the "yielding" system or as an elastically responding system. As capacity loads are hard to predict

it is recommended that brittle components be designed to the limit of their structural ductility factor when brittle failure occurs (the value recommended for design purposes is $\mu = 1.5$). Reference to NZS 4203 shows that the design loads for $\mu = 1.5$ are twice the design loads for $\mu = 4$. Thus, if brittle elements within a piled house foundation system are designed to a load level equivalent to $\mu = 1.5$, they will not fracture until the house lateral load is twice the ductile, ($\mu = 4$) design load. Even though a house may be much stronger than required by SANZ (1990a), at twice the design load a significant amount of slackness will be present from a combination of wall, foundation connection and pile/soil interaction yielding. Thus, for instance, from Table 4.8 of NZS 3604, an anchor pile can be assessed as having maximum earthquake resistance of $2 \times 70 = 140$ BU force at the top for earthquake compared with 160 BU for wind. Thus wind governs for the capacity design of the pile timber and pile top connections.

The draft Timber Pile and Pole Standard DZ 3605 (SANZ 1992b) intends that 95 percent of 140 mm diameter and 125 mm square house anchor piles have a bending moment strength of at least 7.2 kNm. This implies a short-term timber flexural stress of 26.7 MPa for round piles and 22.1 MPa for square piles. There are four methods whereby pile suppliers can show compliance with the strength requirement of DZ 3605. These includes batch testing to 7.2 kNm, proof loading all piles to a bending moment of 6 kNm, and two alternative visual grading criteria. This report assumes the ultimate limit state pile bending moment of 7.2 kNm is controlled by earthquake conditions and that a ductility factor of 1.5 is applicable for reasons discussed in the above paragraph. For 1.2 m high, driven piles this results in an earthquake design strength of $7.2/(1.2 + 0.3)/2 \times 20 = 48$ BU or 1.6 times the current bracing rating used in NZS 3604. (It is assumed that the maximum bending moment occurs 300 mm below the soil surface level.) For a laterally loaded, 600 mm high anchor pile the earthquake design strength is $7.2/0.6/2 \times 20 = 120$ BU or almost twice the value stipulated in Table 4.8 of NZS 3604. This assumes that the anchor pile timber failure occurs at the soil surface, as at greater depths the required pile concrete surround strengthens the pile.

2.6 Test Procedure for Foundation Systems

2.6.1 Standard (NZS 3604) Strength Requirement for Connections between Floor and Piles

To ensure that the design forces can be transmitted from the floor diaphragm to the braced or anchor pile system, NZS 3604 (SANZ, 1990a) requires a 12 kN connection between joist and bearer, and between bearer and pile. This 12 kN connector capacity is back calculated from the 17 kN brace at 45° slope (see Section 2.1.1).

2.6.2 BRANZ Proposed Pile Foundation Connection Test Procedure

Various systems (proprietary and other) have been proposed to achieve the 12 kN connection requirement of the Standard. To date no recognised test procedure has been available to demonstrate compliance with these code provisions. As part of this project, a draft BRANZ Technical Recommendation, "Foundation Test and Evaluation Procedure", was developed to provide a verification method whereby systems could demonstrate that they possessed the required strength (see Appendix A). Three different proprietary systems have been tested by manufacturers of timber pile connection systems using this procedure and BRANZ has observed some of the tests of each of the three systems. Aspects of the procedure need verifying; this is one of the purposes of the experimental work described in this report. For confidence that all systems passing the test criteria will perform adequately in practice it must be shown that (a) pile rotation that occurs in practice (but is restrained in the test) will not substantially affect the performance, and (b) mixed systems of both stiff and flexible elements will still perform satisfactorily.

Thurston (1992) has developed and tested simple methods of resisting the 12 kN force from the floor diaphragm level to bearer level, in the direction perpendicular to the joist. The systems recommended were intended to be inexpensive and practical. They include blocking and the use of construction material normally readily available on a building site.

The procedure in Appendix A derives resistance ratings for foundation systems complying with NZS 3604 (SANZ, 1990a). Although written for pile foundation systems, the principles can readily be applied to any other foundation system. The test procedure stipulates that the test pile is restrained against rotation, and that firstly serviceability and then ultimate limit state deflections, be applied at the floor level. A minimum of three tests is required. The bracing rating evaluated may be governed by either limit state, but usually it is the ultimate limit state which controls. The wind design load is deemed to be the lower 5 percentile PEAK load resisted whereas the earthquake design load is the RESIDUAL load after 4 cycles factored by $F1 \times F2$ as defined in Equation 1. It is recommended that the lower bracing value of either the connection (as tested above) or the pile (as stipulated in NZS 3604) be adopted. Thus, for a ductile connector (i.e., $F2 = 1$) the residual lower 5 percentile earthquake load need only be $70/20 \times F1 = 3.5 \times F1$ kN (rather than 12 kN) to use the current full braced or anchor pile bracing rating given in NZS 3604. Recommended values to be used for $F1$ are provided in the section below.

2.6.3 Effect of Shape of Hysteresis Loops and $F1$ Factor

Dean et al. (1986) performed a series of computer analyses of single degree of freedom models of varying initial stiffnesses and for a variety of ground acceleration earthquake records. They compared the displacement demands for "equivalent" elastoplastic and "pinched" (S-shaped) hysteresis loop models and found that the maximum computed displacements were not significantly different for the two. The yield forces used by Dean et al. were (an unspecified amount) less than the ultimate loads. This effectively allowed the pinched system to have some reserve strength over the elastoplastic system. However, further computer runs showed that a system without this reserve still achieved similar results to an elastoplastic system for the El Centro 1940 earthquake for building periods greater than 0.5 seconds. Similar results were reported by Dean et al. (1987, 1989). It was concluded that for most timber structures, similar deflections resulted in earthquake simulation with models incorporating an initial slackness and "pinched" loops as with elastic loops; secondly it was far more important that structures be able to undergo large displacements without collapse than have high energy absorption capacity as represented by "fat elastoplastic loops".

Dean and Lapish (undated) produced a design guide for multi-storey gypsum plasterboard buildings outside the scope of NZS 3604:1990. They used a computer simulation technique applied to a single degree of freedom oscillator to determine the characteristics of elastoplastic hysteresis loops that provided the same maximum deflection as "pinched (S-shaped) loops. Their analysis indicated that the equivalent elastoplastic yield load (F_y) was about half the peak resisted load from the pinched model. In this analysis they defined the yield displacement (i.e., $\mu = 1$ displacement) to be slightly less than the intersection of the F_y line and the pinched loop backbone curve, (defined by the linear extrapolation of a point on the backbone curve at $0.75 F_y$ to F_y). By using time-history computer analysis techniques, Dean (1987) and Dean and Tjondro (1988) showed the equivalence described above was conservative for a wide range of modelled earthquakes for buildings up to 4 stories high. As the behaviour is more determined by the residual load-deflection behaviour than by the original backbone curve characteristics, for the foundation evaluation in Appendix A an equivalent yield plateau of 0.75 times the lower 5 percentile residual peak load (R) is used. The elastic response 1 displacement $\mu = 1$ is defined as being 1.5 times the deflection on the lower 5 percentile backbone curve at a load of $0.5 \times R$. These definitions of equivalent yield force and ductility imply that the initial elastic portion of the equivalent elastoplastic curve passes through the backbone curve at $0.5 \times R$, as plotted in Figure A.2.

Dean and Lapish (undated) used a wind (ultimate limit state) design load which was between 30 and 40% of the peak resisted load. The basis for this was not stated, and may have been serviceability criteria, but the values appear low when compared to that proposed in Appendix A.

Dean (1992) analysed many different slack systems (i.e., "pinched" (S-shaped) hysteretic curves) with several different earthquake records through a modified version of the computer program discussed above (Dean et al., 1986). He was able to relate their performance to equivalent elastoplastic systems by considering several basic parameters extracted from the pinched loops (see Figure 2). He found that these parameters dictate the earthquake performance, and the shape of the initial loading curve (for instance) had little effect. Eventually, a foundation and light timber framed wall evaluation procedure may be developed, based on Dean's work (1992), in which bracing ratings are determined directly from the parameters measured from test hysteresis loops. This eliminates the need to use F1 and F2 factors, which are difficult to derive (and of a somewhat dubious nature). However, as an interim measure it is recommended that the earthquake foundation bracing rating be derived as outlined in Appendix A, i.e., a F1 factor of 0.75 be used as discussed above. The writer is making further investigation into this factor in a current research study.

2.6.4 Comparison of Proposed Foundation Test Procedure with BRANZ P21 Wall Bracing Test Procedure

Whereas the BRANZ P21 wall bracing test procedure (King and Lim, 1990) evaluates the results for earthquake load based on average test loads and assumes $F1 = 1$ ($F1$ defined in Equation 1), the foundation test procedure proposed in Appendix A assumes $F1 = 0.75$ and derives the design loads using the lower 5 percentile values from the test results. The two procedures differ in the manner that ductility is defined. The less conservative wall bracing procedure recognises that many "non-structural" elements, which are ignored in the design calculations, will in fact enhance the wall bracing strength significantly. This additional "non-structural" bracing generally won't be present in the foundations. Both evaluations assume that the distribution of bracing elements within the house and the compatibility of stiffnesses of the various bracing elements do not unduly increase the loading demand on any particular element. This assumption needs verifying, particularly for foundation elements where element stiffnesses may vary markedly.

The P21 procedure bases the wind resistance load on 0.9 times the average peak resistance assessed through testing, whereas the foundation procedure uses 0.9 times the statistically assessed lower 5 percentile resistance. The reduction factor of 0.9 is intended to account for the strength reduction which may occur when a fluctuating uni-directional load is applied as would be realistic for turbulent wind simulation.

2.6.5 Test Procedure for Fixed Head Piles and Foundation Walls

This section covers situations where the test procedure in Appendix A is applicable to foundation systems other than where the pile is connected to the bearers and the pile is assumed to be rigidly held in the ground. The required adjustments to the test procedure are discussed.

House piles can be effectively flexurally restrained above the soil with a cross brace or sheet material as shown in Figure 3. For these two situations the pile can reasonably be assumed to be "pinned" at the pile mid-depth within the soil, and the soil need only provide shear force resistance associated with this "pin" connection rather than a full flexural restraint. The overturning moment from the applied loads is resisted by axial forces (upwards on one pile and downwards on the other in Figure 3) being generated within the pile pair ("portal action"). In practice the uplift loads may be resisted by pile withdrawal skin friction forces as well as the gravity weight of the house above. Calculations show that the pile uplift force will not exceed the resisting force for any significant period of time, even for corner piles. A small amount of uplift (i.e.,

foundation rocking) is an acceptable energy dissipating mechanism. Thus, it is recommended that full uplift restraint be provided in foundation tests of these systems. For wall bracing tests, the wall uplift resisting mechanisms for bracing elements adjacent to wall openings and corners are not as rigid or strong, and the BRANZ P21 procedure (King and Lim, 1990) limits the level of uplift restraint provided in the test procedure. However, full house racking tests by Reardon (pers. comm. 1992) suggest that full restraint is usually present for wall bracing elements and it is reasonable to provide full restraint during evaluation laboratory tests. The appropriate level of vertical restraint is currently being investigated by BTL.

The maximum soil shear forces are calculated in Table 1, assuming a maximum soil lateral bearing stress of 100 kPa and the minimum dimensions of the piles in the soil as required by NZS 3604:1990 and shown in Table 1. It is recommended that design values for earthquake are factored by $F1 \times F2$ (see Equation 1) with $F1 = 0.75$ and $F2 = 1.0$. The relatively high value of $F2$ was chosen because potential shear failure of the soil is considered to be an acceptable mechanism, as the deformation will be highly damped and easily repaired. Friction forces at the bottom of the piles have been conservatively ignored.

TABLE 1 Maximum Shear Forces that can Theoretically be Resisted by Piles for Cohesive Soils of Lateral Bearing Strength 100 kPa

Pile Type	Overall Concrete Dimensions (mm)		Maximum Soil Resistance Shear Loads (kN)	
	Depth	Diameter**	Earthquake	Wind
Ordinary Pile	300	200*	4.5	6.0
Shallow Cant. Pile	450	230	7.7	10.3
Braced Pile	450	400	13.5	18.0

• No concrete first 100 mm of pile depth.

** Diameters were obtained from Table 4.5b NZS 3604:1990. A square footing has sides slightly smaller than the diameter shown, but is expected to induce higher soil resistance.

A common foundation system that may require testing is where a sheeting foundation wall is constructed upon a masonry foundation wall. In this instance the bracing strength of the system can be determined from the lower of (a) the bracing system for the masonry wall specified in NZS 3604:1990, or (b) the test results of the sheeting foundation wall, tested to the regime described in Appendix A, including the base connection detail to the masonry wall. For testing, the masonry wall can be replaced by a steel beam or other rigid member.

2.7 Relationship Between Building Period, Ductility and Seismic Deflection

The theoretical relationship between the above parameters is plotted in Appendix C. It is based on NZS 4203 (SANZ, 1992b) design spectra and assumes a single degree of freedom (SDOF) system. At a 0.4 second period, the displacements of the SDOF system under the SANZ (1992b) spectra are 26, 31 and 35 mm for ductilities of 1, 2 and 4, respectively. Note, that the loads for foundation elements in NZS 3604:1990 were based on period $T = 0.4$ seconds. Because the earthquake response spectra in the Loadings Standard remain constant for values of T less than 0.45 seconds, the above deflections can not be taken as the minima for the test regime ultimate deflections. A minimum deflection of 16 mm has, however, been stipulated to ensure that if a variety of different bracing systems are used, stiffness incompatibility does not cause overloading and premature failure of any one particular element.

At periods of 0.6 seconds, the displacements of the SDOF system are 51, 55 and 58 mm for ductilities 1, 2 and 4, respectively. Note, that the loads for the bracing wall elements in NZS 3604:1990 were based on $T = 0.6$ seconds.

The relationship between building deflection and seismic load coefficient is also plotted in Appendix C. For any given ductility (μ), a decrease in building stiffness increases the building period, reduces the design seismic coefficient and increases the peak deflection (D_{MAX}) under the design earthquake. However, it requires a large increase in deflection to make significant changes in seismic coefficient. For example, as shown in Appendix C, to obtain the same seismic coefficient for a ductility 2 and a ductility 4 building, the building stiffness must be reduced by a factor of 5.5; the deflection D_{MAX} then increases from 35 to 94 mm under the design earthquake. For an elastic system this deflection is 190 mm. Even if an elastic system (ductility 1) deflects 91 mm the seismic coefficient is still twice that assumed in the Standard. Thus, an elastic system does not obtain a large reduction in earthquake load by a reduction in stiffness and an ability to sustain large deflections. Increasing the ductility is a more efficient process.

3.0 TEST PROGRAMME

3.1 Test Outline and Equipment Used

To investigate aspects of the foundation test and evaluation procedure presented in Appendix A and to study the earthquake and wind loading performance of some common pile systems used in New Zealand houses, the following test programme was initiated.

A shallow cantilevered pile was bolted to a bearer. Yielding steel plates at the base simulated soil yielding conditions. Cyclic horizontal loading was applied to the top of the pile to verify that the connections performed adequately. Two tests were then performed by changing the top connection between pile and bearer to different proprietary systems. BRANZ had been involved in testing both of these systems for compliance with the test procedure outlined in Appendix A, in which piles are fully restrained against rotation. The current testing was designed to determine whether pile rotation will have a significant effect on the results, and whether the top fixing rigidity of the proprietary systems increases the pile bracing strength.

Two bolted braced pile arrangements were then tested to investigate the adequacy of details defined in NZS 3604:1990. The brace was connected between the base of one pile and the top of the other in one test and between one pile base and the bearer in the other test. In both instances the bearer was parallel to the brace. It had been intended to also test configurations where the brace was parallel to the joists and where proprietary top connections were used. However, the low loads obtained from the above tests clearly indicated that the same limiting mechanism and failure load would result, making these programmed tests of little value.

Cyclic loading at 0.25 Hertz was applied to the specimens described below using a 100 kN closed loop electro-hydraulic (Moog) ram. Load was measured by means of a 100 kN load cell; the equipment being within BS 1610 (BSI, 1985) Grade 1 accuracy. Linear potentiometers (reading to an accuracy of 0.25%) were used to measure the various deflections shown in the test set-ups detailed below.

The test load and displacement readings were recorded using an IBM compatible PC running a software program to record data in real-time mode.

3.2 Shallow Cantilevered Piles Bolted to Bearers

The test set-up used two 125 mm square piles coach bolted to two 150 x 50 mm bearers as shown in Figure 4. Washers were used only at the nut end. This is common practice although not strictly allowed by NZS 3604:1990. The pile was scarfed by 75 mm which, although exceeding NZS 3604 recommendation, seems to be common practice. This increased scarfing thickness had little influence on the test results described in this report. The load was applied through blocking at 150 mm above the bearer to simulate a load applied at the floor diaphragm level. The steel plates simulating soil lateral load restraints (hereafter referred to as steel "soil" plates) were located assuming the rectangular stress blocks shown in Figure B.1. This would theoretically separate the restraints from the yield plates by $450/2 = 225$ mm as against the 221 mm dimension actually used. A bearer height of 600 mm above the ground was intended to be modelled. The distance between the bottom of the bearer and the centre of the stress block (Figure B.1) would then be $600 + 450/3 = 750$ mm as against the 768 mm dimension actually used. The system was prevented from buckling out of plane by using a roller system against the bearer; the piles were prevented from lifting by a bolted connection to the strong-floor shown in Figure 4.

Calculations in Appendix B (using rectangular stress blocks - maximum 100 kPa) showed that 0.6 m high cantilevered piles will resist 1.61 kN horizontal load applied at the top. This translates to 3.22 kN for the twin piles in Figure 4. The bracing values from NZS 3604:1990 for two shallow cantilevered piles are 1.2 kN and 3.0 kN for earthquake and wind, respectively. Thus, the calculated resistance is close to that provided by the Standard for wind loading. The steel "soil" plates were selected to match the wind peak load forces.

3.3 Shallow Cantilevered Piles Fastened to Bearers with Type L Connectors

The piles in Figure 4 were truncated just below the pile scarf and a 240 x 100 x 1 mm perforated nail plate (Type L) was used to join the piles to the bearers as shown in Figure 5. Ten 3.15 mm x 30 mm pan head nails were used both above and below the bearer/pile junction in each plate using an approximately circular pattern. The devices used were part of a proprietary system which has a current BRANZ Appraisal for providing a 12 kN foundation connection.

3.4 Shallow Cantilevered Piles Fastened to Bearers with Type T Connectors

The Type L connectors were removed and the bearers repositioned above the piles and reconnected using Type T proprietary connectors. These were assembled according to the manufacturer's instructions and had twelve 3.5 x 35 mm pan head nails between plate and pile as shown in Figure 6. The devices used were part of a proprietary system which has a current BRANZ Appraisal for providing a 12 kN foundation connection.

3.5 Braced Piles - Brace from Pile to Pile

The test set-up used two 125 mm square piles coach bolted to two 150 x 50 mm bearers as shown in Figure 7. Coach bolts were also used on a brace at approximately 45° to the horizontal. Washers were used only at the nut end. The pile was scarfed by 75 mm, exceeding the NZS 3604 recommended scarf depth of 25 mm. The load was applied through blocking at 150 mm above the bearer to simulate load at the floor diaphragm level. The steel plates simulating soil lateral load restraints were positioned assuming the rectangular stress blocks shown in Figure B.1. This theoretically separates the restraints from the yield plates by $450/2 = 225$ mm, as against the 221 mm dimension actually used. The system was prevented from buckling out-of-plane using wire lightly tensioned to a distant support on either side of each pile. This proved to be effective in resisting out-of-plane sway and provided negligible in-plane resistance for the deformations imposed during testing. (This was easily verified by applying transverse deflections by hand at mid-length of the wire.)

Piles were prevented from lifting during loading by the use of tie rods fastened between the strong floor and load skates located directly above the bearer (see Figure 7). This simulates the weight of the rest of the house which would be transferred to the bearer through various framing members, should the bearer lift under house lateral loading.

3.6 Braced Piles - Brace from Pile to Bearer

The original brace was removed, the bearer lowered and moved along the pile, and a new brace was bolted between pile and bearer as shown in Figure 8. The brace was at 41.6° to the horizontal. The remaining set-up was the same as in Section 3.5.

4.0 TEST RESULTS

4.1 Cantilevered Piles

The system was first assembled without the steel "soil" plates and sinusoidally displaced at 0.25 hertz for 4 cycles at amplitudes 20, 40, 60, 80 and 100 mm. As the system was effectively pinned, both at the bolt between pile and bearer and at the pile lateral restraint location, it was expected that there would be little load resistance. The measured load deflection response was essentially linear between zero and 100 mm displacement, when the resisted load was 0.7 kN. This small load is attributed to a small moment restraint generated at the bearer seating on the pile. This was a "couple" with forces at the bolt and edge of the pile.

Load plates (60 x 12 mm) were then installed and the cycling shown in Figure 9 applied. The peak load at 60 mm displacement was 3.85 kN which is slightly higher than the target value of 3.2 kN from Section 2.1. The plates were straightened and, on re-testing similar results were generated to those in the initial test. When the specimen was dismantled, some elongation of the bolt hole was observed in both bearer and pile.

4.2 Shallow Cantilevered Piles Fastened to Bearers with Type L Connectors

Load deflection curves for the system with 60 x 12 mm steel "soil" plates and Type L connectors (see Appendix D) between pile and bearer are shown in Figure 10. The Type L connectors showed little sign of distress after completion of this phase of testing. The stiffness/strength enhancement added by the Type L connectors (due to pile/bearer fixity) can be seen from the comparison of the "spine" curve from the pile without top Type L plates (from Figure 9) which has been reproduced in Figure 10. It was concluded that the Type L plates enhanced significantly the bracing capacity of the system. If the amount of strength enhancement can be verified by the test regime shown in Appendix A for both loading directions, then the bracing rating resistance of cantilevered piles with these fixings can be significantly increased above that provided by NZS 3604:1990. To check whether pile rotation would seriously impair the Type L connection strength, the soil plates were replaced with 200 x 12 mm plates. The Type L plates were not replaced as little damage was observed during testing. The resisted load increased significantly (Figure 11) and this is a function of the steel "soil" plate stiffness/strength. After 4 cycles to 60 mm nail withdrawal was observed; during 90 mm cycles the plates could be seen slipping significantly relative to the timber as the nail holes elongated. However, even at these extreme displacements the plates held on tenaciously without a significant reduction in peak load. The average peak load of 17 kN was more than the wind design load (NZS 3604:1990) for two anchor piles or cantilevered piles (16 kN), and a lot higher than the corresponding earthquake load (7 kN). Although it would have been useful to insert even stronger soil plates to verify that the system still behaved well at 2 x 12 kN, it is still concluded that the system would have performed well in practice and that the effects of pile rotation were not severe.

4.3 Shallow Cantilevered Piles Fastened to Bearers with Type T connectors

The pile system with Type T connectors (see Appendix D) was tested with two strengths of "soil plates". With the 200 x 12 mm plates, similar (but slightly higher) strengths were obtained with the Type T top connection (Figure 12) than with the Type L top connection (Figure 11). At high deformations, the legs of the Type T connection (Figure 6) were buckling above the compression face and nails were pulling slightly out of the main body connection onto the pile. The distress was not considered extreme. With the stiffer double plates the resisted loads increased (particularly in the applied pull rather than push direction (Figure 13)), and a resisted load of more than 2 x 12 kN was recorded. The reason for the lack of symmetry in load resistance was not determined. Although nail pull-out and plate distortion were severe, the Type T connection hung on tenaciously with little load reduction on repeated cycling. It would appear that this connector for this load direction is not unduly affected by pile rotation and (for the short cantilevered pile) would enhance the bracing strength due to top fixity.

4.4 Braced Piles - Brace from Pile to Pile

The pile system was first loaded without the brace to identify the load resisted by the plates alone. The results are given in Figure 14. A repeat test with straightened plates gave almost identical results. After bolting on the brace the hysteresis loops shown in Figure 15 were produced. Also plotted on this graph is the backbone envelope from Figure 14. The strength enhancement due to the brace averages 3.8 kN at 60 mm. This is lower than the 12 kN assumed when deriving NZS 3604 brace design loads (SANZ, 1990a). The main movement observed was attributed to the slip between brace and pile, as was detected from the following measurements. After 40 mm cycling, the specimen was pushed to 40 mm, and held, and then pulled to 40 mm and held. The slip of the brace relative to each pile was measured from scratch-marks each time. An average value of 7 mm occurred in the direction of the brace. The horizontal slip between piles and bearer was also measured and averaged 3 mm. The corresponding values at 100 mm static deflections were 20 and 4 mm, respectively. The bolt elongated the hole, approximately equally in both members, at the interface between pile and brace in the direction of the brace. The depth that this elongation penetrated into the timber was approximately equal to elongation length.

4.5 Braced Piles - Brace from Pile to Bearer

The pile system was first loaded with a packer being used between pile and brace and then with this packer removed. The results are given in Figure 16 and 17, respectively. Again, the system was very flexible and resisted loads were low. Values were very similar to that in Section 3.4. The main movement was attributed to the slip between brace and pile as was detected from static measurements. The brace slip measured 6 and 21 mm at static deflections of 40 and 100 mm, respectively, as discussed in Section 4.4. The corresponding values for the slip between pile and bearer were 2 and 5 mm. The brace bolt elongated the hole at the interface brace/connection as above.

5.0 DESIGN VALUES

There has not been sufficient experimental and theoretical work to recommend revised foundation design values appropriate to NZS 3604:1990. However, based on the preceding sections an estimate of what are likely values are derived below although it is recognised that they need verification by field tests. The factors used to downgrade the estimated strengths to design values were discussed in Section 2. Areas of future research are highlighted. The data given below are indicative only and must not be taken as design values

until verified by further research and adopted by Standards New Zealand. In particular, the influence of cyclic load on the lateral strength of squat piles is not well known, nor the performance of squat piles in sand.

5.1 Diagonal Braces Fixed With M12 Bolts

This category covers braces between two piles, or pile/bearer, or pile/joist or sub-floor framing. The testing indicated that the load limiting mechanism with these systems was likely to be the bolted connections at each end of the brace. The Timber Design Standard NZS 3603 (SANZ, 1990b) allows a working stress design load of 2.65 kN per bolt loaded at 45° to the grain (short-term loading, single shear). This can be factored by 1.69 (Walford, 1992) to give an ultimate load of 4.5 kN per bolt. This would result in an ultimate horizontal strength of 3.2 kN for a 45° brace. Test results for bolt ultimate strengths are significantly higher than NZS 3603: 1990 values. After 4 cycles at 60 mm horizontal deflection of a braced system, the testing described in this report measured the horizontal load attributed to the brace as 3.8 kN. The strength was 5.8 kN if the steel "soil" plates are included. However, this was one test and the results should be downgraded by about 25% to approximately provide a lower 5 percentile strength. A reduction of a further 25% (i.e., $F1 = 0.75$) is also made as the hysteresis loops are "pinched" rather than elastoplastic, as recommended in Section 2.6.4. The earthquake design bracing strength, BU(EQ-Brace) is calculated in Equation 2. This assumes the brace can provide a ductility of 2 at 60 mm displacement. The seismic coefficient of 0.356 was obtained from Figure C.1(b) at this deflection, while the coefficient of 0.27 was from SANZ (1992b) for $\mu = 4$ and $T=0.4$.

$$BU(EQ-Brace) = 5.8 \times 0.75 \times 0.75 \times (0.27)/(0.356) \times 20 = 50 \text{ BU} \dots\dots (2)$$

This value is well below the NZS 3604:1990 design value of 70 BU. The corresponding value for wind is 116 BU as against 160 BU allowed by NZS 3604:1990. The test values should be considered "worst case" and if the soil strength is above the minimum allowed by NZS 3604 then higher bracing values would result.

5.2 Anchor Piles

Tests are required to determine the lateral strength of anchor piles in soil. Using fairly conservative assumptions, calculations in Appendix B indicate a lateral strength of 7 kN. This translates to a wind bracing value of 140 BU as against 160 BU provided by NZS 3604:1990. A soil failure mechanism is considered to be an acceptable behaviour as the deformation will be highly damped and easily repaired. Based on $F1 = 0.75$ and $F2 = 1$ the earthquake capacity would be 105 BU as against 70 BU provided by NZS 3604 - although this needs to be verified by examination of test hysteresis loops. Calculations in Section 2.5, using a capacity design approach shows that pile fracture need not be a concern with these design values.

5.3 Shallow Cantilevered Piles

Tests are also required to determine the lateral strength of shallow cantilevered piles in soil. It is noted, however, that these are likely to be highly dependent on soil properties and pile installation technique. Using fairly conservative assumptions, calculations in Appendix B indicate a lateral strength of 0.92 kN. This translates to a wind bracing value of 18 BU as against 30 BU provided by NZS 3604: 1990. Based on $F1 = 0.75$ and $F2 = 1$ the earthquake capacity would be 14 BU as against 12 BU provided by the Standard. However, ignoring friction on the bottom of the pile is likely to be very conservative.

5.4 Deep Cantilevered Piles

NZS 3604:1990 specifies design values of 3.5 kN for wind and 1.5 kN for earthquake for these piles. Based on pile flexural failure, calculations in Section 2.5 show the approximate design loads are 4.8 and 2.4 kN for wind and earthquake, respectively. Section 2.3.2 discussed extensive tests of these piles in weak Auckland soils. The average test loads at pile surface level deformation of 20 mm were 8.2, 6.9 and 5.4 kN for soil Scaia Penetrometer strengths of 3-4 blows/75 mm, 2-3 blows/75 mm and 1-2 blows/75 mm, respectively. (The latter two soil groups were too soft to satisfy NZS 3604:1990 minimum soil strength criteria.) However, for all three groups the measured average pile lateral resistance at 20 mm deformation exceeded the wind load based on pile flexural capacity.

5.5 Diagonal Bracing Fixed to Sub-floor Framing

The Standard design load Table 4.8 of NZS 3604:1990 is 1.1 kN for earthquake and 4 kN for wind. NZS 3604:1990 requires the fixings at each end of the brace to be able to resist 8.5 kN in the direction of the brace. This equates to a horizontal strength of 6 kN. This has been downgraded by 50% (to take into account variability of construction) to obtain the 4 kN Standard wind design strength. To obtain the earthquake bracing rating a capacity design approach is recommended, (i.e., assume a ductility of 1.5 and hence divide loads by 2 as outlined in Section 2.5). Thus, the earthquake design load becomes $4 \text{ kN}/2 = 2.0 \text{ kN}$ rather than 1.1 kN as specified by NZS 3604:1990.

6.0 CONCLUSIONS

Use of published formulae to calculate pile strength and deflection gives a wide range of results depending on the soil properties assumed. The properties, applicable to the minimum strength soil allowed in construction to NZS 3604:1990, must be used. Analytical methods examined in this report assume pile shape is less squat than used in New Zealand house construction, and only give broad guidelines on the soil strength characteristics over depths equal to the first few pile diameters. A literature survey and experimental programme to determine the lateral resistance of short piles in the minimum strength soil complying with NZS 3604:1990 is necessary to provide confidence in the design procedures currently being used; these are quite conservative.

Rigid pile deflections were calculated using published formulae based on soil subgrade reactions. The predictions show that pile deflections in the soil are not critical in stiff over-consolidated clays, but will govern in soft, normally consolidated clays. Deflections will be small in sands not subject to liquefaction.

The foundation test procedure recommended in Appendix A requires serviceability cyclic deformations to be imposed at deflections of up to 8 mm. It is recommended that houses be tested at service wind and earthquake loads to determine acceptable service-load deflections regarding human comfort and onset of structural damage to the building and services.

Deflections of up to 60 mm under ultimate wind or earthquake loads are specified in the foundation test procedure recommended in Appendix A. It is recommended that a study be made at these large deflections to ascertain the likely magnitude of damage to secondary elements, the associated repair costs, secondary damage effects, loss of function costs etc. The results of this study may limit maximum deflections allowed in tests.

For the proprietary systems tested, pile rotation, (restrained in the recommended procedure in Appendix A), did not appear to significantly detract from the performance of the foundation connection systems.

At this stage it appears that it would be better to alter the P21 procedure for determination of wall bracing ratings to enable the evaluation of bracing rating directly from the test results (hysteresis loops) based on Dean's work (Dean, 1992). This would eliminate using F1 and F2 factors, which are difficult to derive (and of a somewhat dubious nature) for "pinched S-shaped" hysteresis loops.

On the basis of test results, foundation brace design values given in NZS 3604:1990 appear to be too high. Additionally, earthquake design values for shallow and deep cantilevered piles and anchor piles appear to be too low.

Lateral capacities of soils for sheet braced systems (where the pile head is effectively flexurally restrained) are suggested, but need verifying by test. These assume that lateral shear failures of the soil are not an undesirable failure mechanism.

The relationship between house deflection and seismic load coefficient has been derived. At large deflections the fundamental period of the house reduced slightly with a nominal reduction in seismic force as a result. However, even at very large deflections (94 mm) the seismic force attracted to the system is still twice that assumed in NZS 3604:1990 for a ductility = 4 structure. Thus, an elastic system does not obtain a significant reduction in earthquake load by the increase in building period from its ability to sustain large deflections. Providing details to ensure a greater building ductility is a more efficient process.

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APPENDIX A

PROPOSED TEST PROCEDURE FOR FOUNDATION SYSTEMS FIXING DEVICES

S J Thurston and A B King

Introduction

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This is a draft recommendation for a test method and evaluation procedure to determine the structural performance of foundation systems, used in conjunction with light timber framed buildings complying with NZS 3604:1990. Other performance criteria (e.g., durability) are not considered in this document. The evaluation has used a rational approach in accordance with the structural models used to derive the foundation loadings in NZS 3604:1990. This document has been drafted with the evaluation of pile connection details particularly in mind, but may also be used to evaluate the bracing rating of other forms of foundation systems.

Specimen Specification

The manufacturer shall prepare appropriate drawings and associated specifications which detail the components which make up the foundation system, detailing those which are to be supplied by the manufacturer, and any supplementary construction requirements which may be required (above the minimum of those specified within NZS 3604:1990). How components are installed into the foundation system, and any limitations on its intended use, shall be fully detailed.

In general, anchor pile and cantilevered pile systems may be anticipated to accept and transmit loadings in both horizontal directions, i.e., parallel to the axis of both the sub-floor bearer and joist. A separate series of tests will usually be required to demonstrate performance in each of these directions.

Braced piles, sheathed walls or foundation walls will generally accept and transmit load only within their own plane. Although each series of tests will usually only require to be tested in this direction, differences in construction technique (such as the bearer or ribbon plate connection), where lower bound conditions cannot be established, may necessitate individual tests.

The Test Specimens

Each specimen shall include the floor sheathing, not less than two floor joists, each at least 600 mm in length, a length of bearer (at least 800 mm for an internal pile, and at least 600 mm for an end pile) and a pile section of sufficient length to allow full fixity of the pile stub. The sectional dimensions of joist and bearer are to be selected such that the aspect ratio (i.e., depth to width) shall be the maximum for which the results of the test may be applied. Results obtained from nail-spliced bearers may be applied to solid timber bearers of the same overall dimension.

The test specimen size specified above shall be considered to be a minimum size only. The size of test specimens shall not, however, exceed the pile spacing as dictated by the bearer and joist spans published in NZS 3604:1990. The specimens may incorporate the minimum joist blocking at the maximum distance from the pile specified in NZS 3604:1990. The diaphragm shall be constructed of 19 mm flooring grade particleboard. Where full sheets are used in the test specimen, perimeter edge nailing to 100 x 50 mm timber

blocking (on the flat) can be used between the joists. These shall be no closer than 1150 mm from the centre-line of the pile. The provisions of this paragraph can be used to reduce the potential for timber splitting and joist "roll-over."

Selection of Timber

Splitting or fracturing of the timber joists, bearers or piles often occurs in foundation tests performed according to this test procedure. BRANZ believes that this is a valid failure mechanism and the results of these tests should be included in the test evaluation. Thus, timber selection should realistically reflect that used in practice. The test timber shall be randomly selected and the purchased grade shall be specified as being Number 1 Framing Grade (or lower quality) Radiata Pine. Only one portion from each stick shall be used for a component type in any particular test specimen and the portion used shall not be selected from the stick to minimise or reduce the number or severity of timber defects. Thus, a single joist and bearer for each of three specimens can be cut from a stick, but not two joists for a single specimen. The timber components are to be assembled in their wet state, and may be tested in either their wet state or conditioned to a dry state. The moisture content of joists and bearers are to be recorded both at time of assembly and at time of test.

Construction of Test Specimens

Specimens are to be assembled in accordance with the manufacturer's specification. All fixings used are to be typical of those supplied by the manufacturer. The specimen is to be supported by rigidly clamping the pile leg, at least 500 mm below the level of the floor sheathing. Secondary supports are to be provided to ensure that the flooring system remains parallel to its initial alignment thereby avoiding rolling actions or distortions at high displacements. This support shall be such that rotation of either the joist or the bearer is not prevented.

Number of specimens

A minimum of three specimens are required for each configuration in each loading direction. Greater confidence can be obtained by testing more than this minimum number, which is reflected in the application of the statistical procedure required to determine the lower 5 percentile of the results, once the mean and standard deviation are known.

Instrumentation

The following data are required to be measured at each test cyclic peak load and the reduced values are to be tabulated in the test report:

- (a) the magnitude of the applied lateral load;
- (b) the lateral displacement of the floor sheathing immediately above the pile centre;
- (c) the lateral displacement at the bearer/joist interface; and
- (d) the lateral displacement at the pile/bearer interface.

In addition, plots of applied load versus total displacement are to be presented for each test specimen. If the associated data are not continuously recorded then there shall be at least 20 (approximately equal) increments per cycle.

Application of Load

Load shall be applied at the floor sheathing level. Localised reinforcing of the floor sheathing at the connection to the loading device may be required. This is done by attaching plywood or other timber elements to one or both sides of the sheathing as appropriate for the level of loading required. The connection of such stiffeners is to be over-designed to minimise slip at the stiffener/sheathing interface. Stiffeners are not to be connected to any framing members in the assembly.

The minimum loading rate shall be 2 minutes per cycle. The maximum load rate is 0.2 hertz but shall not exceed a rate where at least 20 readings per cycle can be taken. Although the wind and earthquake loading rate will in practice exceed 0.2 hertz this has been taken as the maximum loading rate to allow adequate observation of specimen behaviour. Materials and connections generally have higher strengths at faster loading rates. Excessively slow rates of displacement may tend to produce unnecessarily conservative results through creep effects.

Test Procedure

- A) Establish the displacement associated with the peak load either:
- (a) by calculation, or
 - (b) by applying a uni-directional load (in the least advantageous direction where applicable) until a maximum load level is reached, or
 - (c) by some other method.
- B) Select a suitable serviceability displacement level (d_s) such that d_s is less than or equal to 8 mm. (Typically select d_s to be 8 mm.)

Cyclically displace the specimen four times to a displacement level of $\pm d_s$ (mm). Record the two residual deflections at zero load obtained during the last cycle. Average these two values and label as C. (See Figure A.1).

Label the fourth cycle maximum load encountered in the positive cycle as S+, and the corresponding maximum load encountered in the negative cycle as S-.

- C) Cyclically displace the specimen twice to a displacement level of $\pm d_y$ (mm) for the values of d_y ; starting from approximately 4 mm more than d_s and then incrementally increasing by 4 mm each time until a displacement of $d_u/2$ is exceeded. See D below for definition of d_u .

Label the second cycle maximum load encountered in the positive cycle as Y+, and the corresponding maximum load encountered in the negative cycle as Y- for each value of d_y .

- D) Determine an upper bound displacement level (d_u) such that d_u is less than or equal to d_{max} but not greater than 60 mm or less than 16 mm. Cycle four times to displacement $\pm d_u$.

The evaluation procedure given below penalises loading regimes where d_u is less than $4 \times d_y$. However, for brittle/stiff systems, a higher bracing rating may result if low values of d_u are used and the penalty accepted.

Label the maximum load encountered in the positive cycle as P+, and the maximum load encountered in the negative cycle as P-.

Label the peak positive (residual) load encountered after four cycles R+, and the peak negative (residual) load encountered after four cycles R-.

E) If $d_u < d_{max}$, apply a further single cycle of loading at a selected deflection less than or equal to d_{max} . If higher peak loads are obtained then these loads may be used instead of P+ and P-.

Interpretation of Results

The load carrying capacity of each configuration is considered as the lower 5 percentile of the peak loads obtained from each series of tests. The population of the loads is to be considered to have a log-normal distribution and to be assessed statistically accordingly.

If the peak load in one direction exceeds the corresponding peak load in the other direction by more than 20%, then reduce it to precisely 20% more than the lower value. The peak load for each specimen shall equal the average of the positive and negative maximum loads from the above testing. Thus obtain average values from each test for S (from S+ and S-), Y (from Y+ and Y-), R (from R+ and R-), and P (from P+ and P-). Using the values from all the tests, calculate the lower 5 percentile values; S_{05} , Y_{05} , R_{05} and P_{05} .

Determination of Bracing Ratings

A) For Wind Load Conditions

The theoretical relationship between the peak and serviceability loads (used below) was derived by King and Lim (1991); the theory is repeated later in this Appendix.

The design wind bracing rating is the minimum value derived from Equations A.1 and A.2. These are based on criteria for the serviceability and ultimate limit states respectively. The factor of 0.9 in Equation A.2 is to take into account strength loss from repeated loading due to wind turbulence effects.

$$BU_w = 20 \times S_{05} \times K1/0.65 \quad \dots\dots\dots (A.1)$$

$$BU_w = 20 \times P_{05} \times 0.9 \quad \dots\dots\dots (A.2)$$

where: S_{05} is the lower 5 percentile (as defined above) of all averages of (S+ and S-).

P_{05} is the lower 5 percentile of all peak loads (P) as defined above of all averages of (P+ and P-)

$K1 = 1.4 - C_{av}/d_s$ but is not greater than 1.

C_{av} is the average value of C recorded for all the specimens tested.

If K1 is less than 0.8 then the system is given zero rating. The reason for this is that under serviceability loading a predominantly elastic behaviour is preferred, thereby minimising permanent offsets at this load level. A modest amount of inelastic behaviour is permitted.

B) For Earthquake Conditions

It is necessary to determine the ductility achieved by the system. This is assessed as $\mu = d_u/d_y$. Plot the lower 5 percentile of all the cyclic peak residual loads (S_{05} , all Y_{05} and R_{05}) against the corresponding displacements, joining all points with straight lines as shown in Figure A.2. From this plot, determine the displacement corresponding to the load $R_{05}/2$ and factor this by 1.5 to give d_y .

From a ductility μ found above, a corresponding F2 value is extracted from Table A.1 below using linear interpolation if necessary. F2 is a factor to adjust the test load of the system to reflect the ductility of the tested system compared with the ductility assumed in code loads.

TABLE A.1 Values of F2 for Different Ductility Ratios

When $d_u/d_y =$	1.0	1.25	2.0	2.5	3.0	3.5	4.0 or greater
F2 =	0.35	0.39	0.55	0.64	0.77	0.87	1.0

The design earthquake bracing rating is the minimum value derived from equation A.3 and A.4. These are based on criteria for the serviceability and ultimate limit states respectively.

$$BU_e = (15 \times S_{05}) / (0.49 \times F2) \quad \dots\dots\dots (A.3)$$

$$BU_e = 15 \times R_{05} \times F2 \quad \dots\dots\dots (A.4)$$

where: S_{05} is the lower 5 percentile (as defined above) of all averages of (S+ and S-).
 R_{05} is the lower 5 percentile (as defined above) of all averages of (R+ and R-).

Note: this equates the test hysteresis loops to equivalent elastoplastic loops, where the yield occurs at a force of $0.75 \times R_{05}$ at displacement d_y and the maximum deformation is d_u .

Relationship With NZS 3604 12 kN and 6 kN Connectors

This section considers testing of proprietary connections between piles and bearers and bearers and joists, for which the Standard stipulates 12 or 6 kN connectors. As long as the bracing ratings for systems, as evaluated above, equal or exceed the ratings provided in Table 4.8 of NZS 3604:1990 for diagonally braced and anchor piles, then BRANZ considers that such systems may be used in lieu of the 12 kN connections as defined in the Standard. Similarly, bracing ratings that equal or exceed the ratings provided in Table 4.8, for deep cantilevered piles can be used in lieu of the 6 kN connections defined in the Standard.

In addition, the connectors can be stated as being 12 kN (or 6 kN) connectors if the provisions of the two paragraphs below are satisfied. The concept of ductility is not applicable in this situation; thus, only cycling at displacements d_y is required. The connectors do, however, need to be shown to perform adequately at serviceability loads. The ratio of serviceability to ultimate wind loads = 0.65 (as shown below) with the corresponding ratio for earthquake loads being somewhat smaller. Thus, as NZS 3604 ultimate design wind force per pile = 160 BU (i.e., 8 kN), the serviceability wind force = 5.2 kN. This force is expected to occur about once in 20 years. If the total foundation deflection exceeds about 15 mm then damage to building services is likely. If it is assumed that the deflection of the connector only accounts for about half the total foundation deflection, then a criterion is arrived at, that at a (12 kN) connector load of 5.2 kN the deflection should not exceed 8 mm. The corresponding load for a 6 kN connector = 2.3 kN (from 70 BU / 160 BU x 5.2).

Thus, if the residual lower 5 percentile load, R_{05} , from the above tests exceeds 12 kN and the average first cycle peak load at a deflection of 8 mm exceeds 4.5 kN, then the connector satisfies the intent of NZS 3604 (SANZ, 1990) for being a 12 kN connector.

If the residual lower 5 percentile load, R_{05} , from the above tests exceeds 6 kN and the average first cycle peak load at a deflection of 8 mm exceeds 2.0 kN, then the connector satisfies the intent of NZS 3604 for being a 6 kN connector.

Background to Bracing Ratings

A) Rating Values From Ultimate Load

Assuming a 0.4 seconds period (T) house on "Intermediate" soil, the design earthquake load V_u on a building of weight W and ductility μ , is given by:

$$V_u = C_h(0.4, \mu) \times S_p \times R \times Z \times L_u \times W \text{ from Equation 4.6.2 of NZS 4203 (SANZ, 1992).}$$

Similarly, with a 0.4 second period (T) house, the design earthquake load on a building of weight W and ductility 4 $V_u (\mu=4)$, is given by:

$$V_u (\mu=4) = C_h (0.4, 4) \times S_p \times R \times Z \times L_u \times W$$

Therefore, $V_u/V_u (\mu = 4) = C_h (0.4 \times \mu) / C_h (0.4 \times 4) = 1/F2$ from Table 1.

Therefore, to convert a bracing rating as derived from the NZS 3604 methodology which assumes loads which are based on $V_u (\mu = 4)$ to the correct V_u , factor the bracing rating by F2.

B) Rating Values From Serviceability Load

From Equations 4.6.1 and 4.6.3 of NZS 4203 (SANZ, 1992a), the ratio of serviceability earthquake loads to ultimate loads is given by:

$$V_s/V_u = (C_h(0.4, 1) \times S_p \times R \times Z \times L_s \times W) / ((C_h(0.4, \mu) \times S_p \times R \times Z \times L_u \times W)$$

However, $L_s/L_u = 0.167$ from NZS 4203. Therefore:

$$\begin{aligned} V_s/V_u &= (C_h (0.4, 1) \times 0.167 / C_h (0.4, \mu)) \\ &= (C_h (0.4, 1) / C_h (0.4, 4)) \times (C_h (0.4, 4) / C_h (0.4, \mu)) \times 0.167 \\ &= 0.8 / 0.27 \times F2 \times 0.167 \quad (\text{from NZS 4203 and using F2 as defined above}) \\ &= 0.49 \times F2 \end{aligned}$$

Thus, the earthquake rating is $(V_u/V_s) / (0.49 \times F2)$ if serviceability governs.

The ratio of serviceability to ultimate wind force is directly obtained from NZS 3604 using the ratio of wind speeds specified.

$$W_s/W_u = (0.75/0.93) \times (0.75/0.93) = 0.65.$$

C) Correction For Shape of Load Displacement Curves

Earthquake design loads in this Technical Recommendation were obtained by assuming that "pinched S-shaped" hysteresis loops with residual load R were equivalent to elastoplastic loops with a yield value of $F1 \times R$ where F1 was taken as 0.75. BRANZ is currently investigating this factor and it is subject to change. It is considered that the 0.75 value used is probably conservative.

APPENDIX B

SUPPORTING CALCULATIONS

B.1 Soil Capacity to Resist Anchor Pile Moments and Shear

The equations for resisting soil pressures for the rectangular and parabolic models are given in Figures B.1 and B.2 respectively. NZS 3604 requires the footings to be square (sides 350 mm) or circular (diameter 400 mm). Pile resistance is calculated assuming the pile is 350 mm diameter, although it is recognised that this will provide a lower bound solution.

Anchor pile resistance is plotted in Figure B.3. Based on limiting the upper rectangular stress block to 100 kPa the pile ultimate load is 7 kN. A solution by Broms (1964a, 1964b) in Figure B.4 gives more conservative results for very squat piles.

B.2 Soil Capacity to Resist Short Cantilevered Pile Moments and Shear

The stresses S1 and S2 are both limited to 100 kPa for a shallow pile. For pile depth = 450 mm and pile diameter = 200 mm the lateral resistance can be calculated as 0.92 kN from Figure B.1.

B.3 Pile Deflection Calculations

Poulos and Davis (1980) provide formulae for calculating the deflection (D) and rotation (R) of freehead rigid piles at the ground level. These are all a function of the modulus of subgrade reaction (Kh).

A) Kh Constant With Depth. This is applicable to piles of diameter d in stiff over-consolidated clays. The lower value suggested for Kh x d for stiff clay was 33 T/ft² or 3590 kPa (Terzaghi and Peck, 1967). An alternative value proposed by Poulos and Davis was $67.C_u = 67.50 = 3350$ kPa for soft to medium clay.

$$D = 4H(1 + 1.5 \times e/L)/(Kh \times d \times L) \quad \dots\dots\dots (B.1)$$

$$R = 6H(1 + 2 \times e/L)/(Kh \times d \times L^2) \quad \dots\dots\dots (B.2)$$

Where: H = horizontal load applied at e above ground level to a pile of length L and diameter d.

For an anchor pile with H = 8 kN (i.e., NZS 3604 wind load), e = 0.6 m, L = 0.9 m and Kh x d = 3000 kPa: D = 18 mm and R = 0.035 radians giving a deflection of 39 mm at the pile top.

For a shallow cantilevered pile with H = 1.5 kN (i.e., NZS 3604 wind load), e = 0.6 m, L = 0.45 m and Kh x d = 3000 kPa: D = 13 mm and R = 0.054 radians giving a deflection of 45 mm at the pile top.

Note: as the formulae are independent of pile diameter, they are probably conservative for the squat piles considered.

B) Kh Increases Linearly With Depth (Kh = nh x z/d). This is applicable to soft, normally consolidated soils. An upper range for nh for soft non consolidated clay is 12.7 lb/in³ = 3400 kN/m³.

$$D = 18H(1+1.33 \times e/L)/(L^2 \times nh) \quad \dots\dots\dots (B.3)$$

$$R = 24H(1+1.5 \times e/L)/(L^3 \times nh) \quad \dots\dots\dots (B.4)$$

For an anchor pile with $H = 8 \text{ kN}$ (i.e., NZS 3604 wind load), $e = 0.6 \text{ m}$, $L = 0.9 \text{ m}$ and $nh = 3400 \text{ kPa}$: $D = 74 \text{ mm}$ and $R = 0.116$ radians giving a deflection of 144 mm at the pile top.

For a shallow cantilevered pile with $H = 1.5 \text{ kN}$ (i.e., NZS 3604 wind load), $e = 0.6 \text{ m}$, $L = 0.45 \text{ m}$ and $nh = 3400 \text{ kPa}$: $D = 108 \text{ mm}$ and $R = 0.349$ radians giving a deflection of 317 mm at the pile top.

Note: as the formulae are independent of pile diameter, they are probably conservative for the squat piles considered.

The value of nh for dry or moist medium density sands is approximately $200,000 \text{ kN/m}^3$ (i.e., more than 50 times greater than used above); thus, deflection will not govern.

APPENDIX C

RELATIONSHIP BETWEEN BUILDING DEFLECTION AND BUILDING PERIOD

C.1 Theory

Consider a single degree of freedom (SDOF) house of mass M , stiffness K and period greater than 0.4 seconds located on normal soils in Wellington.

From equation 4.6.2 (a) of the Loadings Standard NZS 4203 (SANZ, 1992a) the design seismic force, V , on the house is given by:

$$V = C_h (T, \mu) \times S_p \times R \times Z \times L_u \times M \times g \quad \dots\dots\dots (C.1)$$

Where g is the acceleration due to gravity (9810 mm/sec²) and Z is the zone factor. Take $R = L_u = 1.0$. $C_h (T, \mu)$ is the seismic acceleration coefficient for period T and ductility μ . The force, V , at the first yield deflection (d) (as defined in Figure C.1) is also given by:

$$V = K \times D \quad \dots\dots\dots (C.2)$$

Standard texts provide a relationship between period T and stiffness K for a SDOF elastic system, viz:

$$T = 2 \pi \sqrt{\frac{M}{K}}$$

$$= 2 \pi \sqrt{\frac{D}{(S_p \times Z \times g \times C_h (T, \mu))}} \text{ from Equations C.1 and C.2}$$

Rearranging gives the deflection at first yield as:

$$D = S_p \times Z \times g \times C_h (T, \mu) \times \left(\frac{T}{2\pi} \right)^2$$

By definition of the ductility, μ , the NZS 4203 relates the maximum earthquake imposed deflection, D_{max} , to the yield deflection, D , by:

$$D_{max} = \mu \times D \quad \dots\dots\dots (C.4)$$

The solution for D_{max} from Equation C.3 and C.4 is plotted in Figure C.1 for periods between 0.4 and 1.0 seconds for Wellington ($Z = 1.2$) and for $S_p = 0.67$. The derivation is based on NZS 4203 purely on published coefficients (which were in turn derived from time history analysis of a SDOF system, using many earthquakes and a uniform risk envelope) and the usual elastoplastic definition of ductility from Equation C.4. It is usually assumed for periods greater than 1.0 second, the principle of equal displacement applies for periods less than 0.2 second that the equal energy principle applies. The applicability of these principles can be checked against the results in Figure C.1. If the equal displacement principle is true, then buildings with the same period (i.e., same initial stiffness K/M ratio) will have the same maximum deflection, irrespective of the ductility. Figure C.1 (a) shows that this is approximately true for periods longer than 0.7 seconds. From figure C.1, at a period of 0.45 seconds, the displacements are 32.1, 39.6 and 43.7 mm for ductilities, $\mu = 1, 2$ and

4, respectively, i.e., at $T = 0.45$ seconds, increasing the ductilities to 2 and 4 from the elastic situation increases the displacements by 22% and 35%, respectively. The equal energy solution (from Figure C.2) gives the corresponding increases as 25% and 112%, respectively. Thus, the equal energy principle gives good agreement with NZS 4203 coefficients for $\mu = 2$ (c.f. 22% and 25%) but poor agreement for $\mu = 4$ (c.f. 35% and 122%). Dean and Buchanan (1987) found a similar result. In recognition of these features, SANZ (1992b) provides a transition zone between 0.2 and 1.0 seconds. For periods less than 0.45 seconds, the Standard provides a constant seismic coefficient; thus, the above ratios will not change.

The Loadings Standard (SANZ, 1992a) seismic coefficients are the same for a $T = 0.4$ second, ductility $\mu = 4$ building (as was used as a basis for NZS 3604 (SANZ, 1990) loads) and a $T = 0.93$, $\mu = 2$ building. This latter building is softer (stiffness reduced by a factor of $(0.93/0.4)^2 = 5.5$) and will deflect 94 mm in the design earthquake as against 35 mm for the ductility = 4 building. This relationship between element deflection and the seismic load coefficient is shown in Figure C.1 (b). An elastic building needs to deflect 190 mm (i.e., period 1.9 seconds) before the seismic coefficient equates to the Standard assumed value ($\mu = 4$, $T = 0.4$). At a deflection of 91 mm the seismic coefficient of the elastic building is still twice that assumed in the Standard. Thus, an elastic system does not obtain a large reduction in earthquake load by having a low stiffness (i.e., increase in building period) and having an ability to sustain large deflections. Increasing the ductility is a more efficient process.

Appendix D

PROPRIETARY PRODUCTS USED

Two proprietary products were used in the experimental programme described in this report and are referred to as Type L and Type T. These products are defined below:

Type L: A 240 x 100 mm Lumberlok plate and nails as described in BRANZ Appraisal Certificate No. 207 1991. Twenty, 45 x 3.55 mm diameter pan head nails were used in each plate, with 10 being located in each pile and bearer.

Type T: A PB2100 Timberlink 12 kN braced pile connection as described in BRANZ Appraisal Certificate No. 187 1990. Twelve, 35 x 3.55 mm diameter dome head nails were used in each plate into the pile, and an additional 4 nails were used in each plate flange and 4 in the plate tongue to connect the connector to the bearer.

Note: Results obtained in this study relate only to the samples tested, and not to any other item of the same or similar description. BRANZ does not necessarily test all brands or types available within the class of items tested, and exclusion of any brand or type is not to be taken as any reflection on it.

Further, the listing of trade or brand names above does not represent endorsement of any named product or imply that it is better or worse than any other available product of its type. A laboratory test may not be exactly representative of the performance of the item in general use.

This work was carried out for specific research purposes, and may not have assessed all aspects of the products named which would be relevant in any specific use. For this reason, BRANZ disclaims all liability for any loss or other deficit, following use of the named products, which is claimed to be caused by reliance on the results published here.

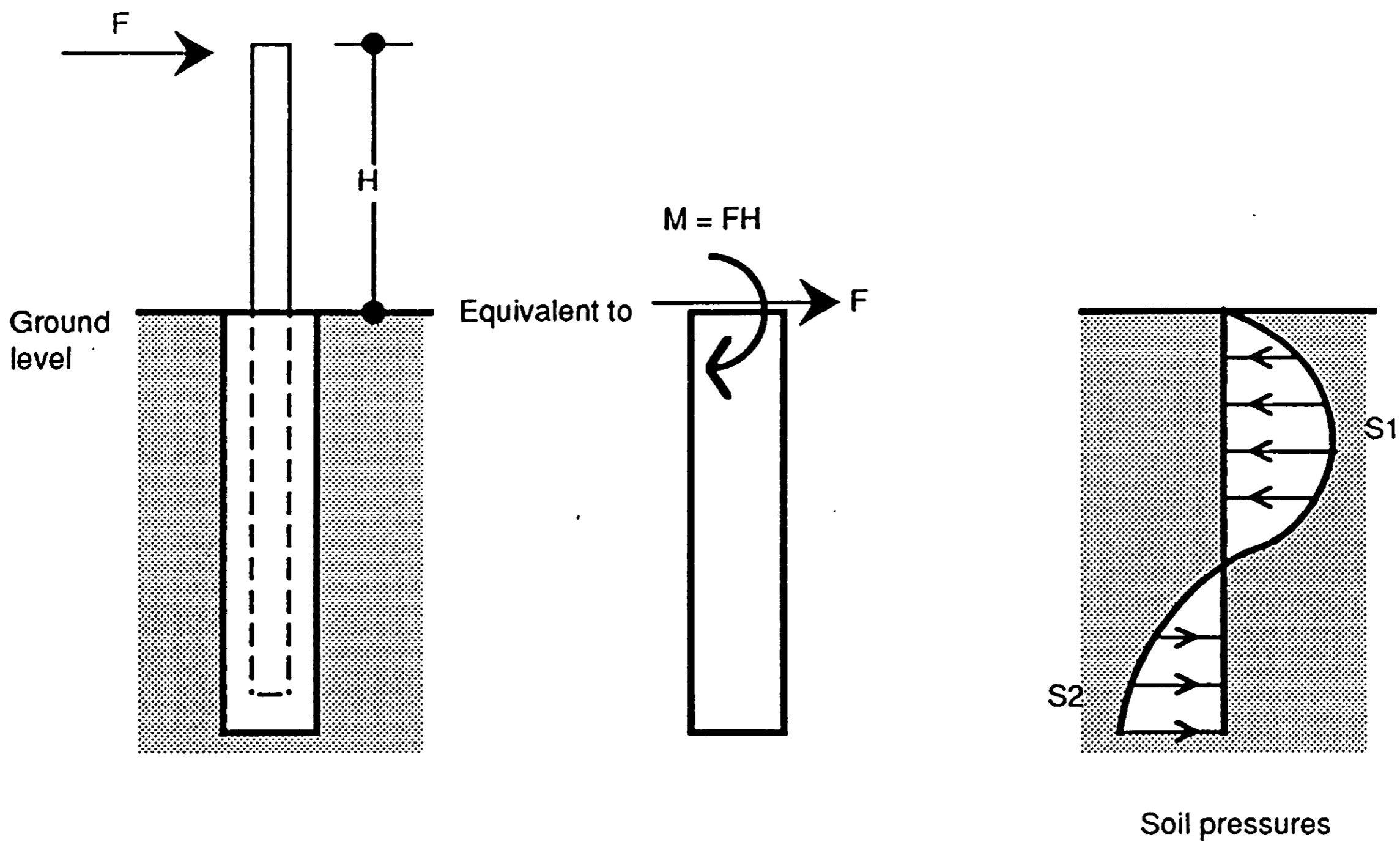
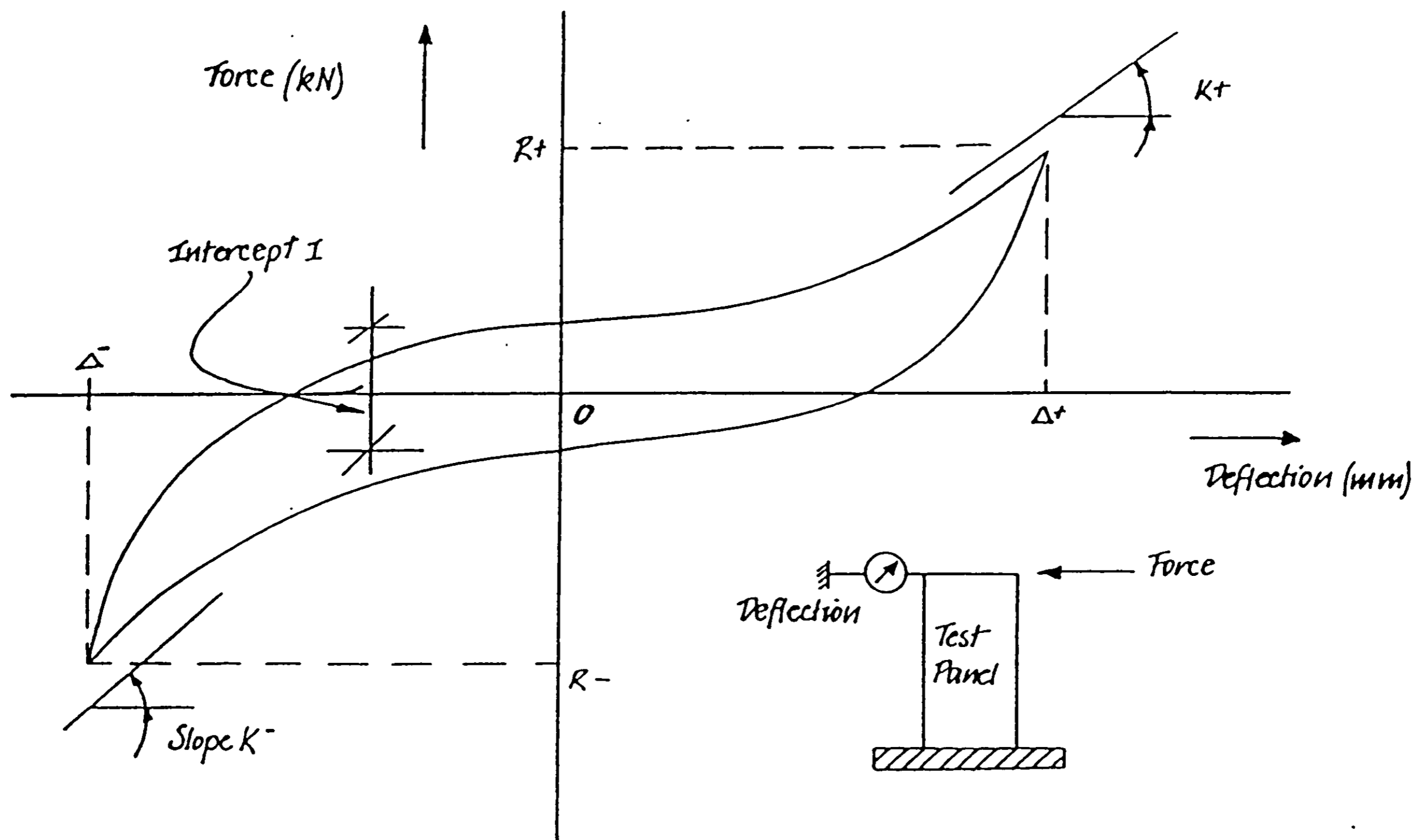


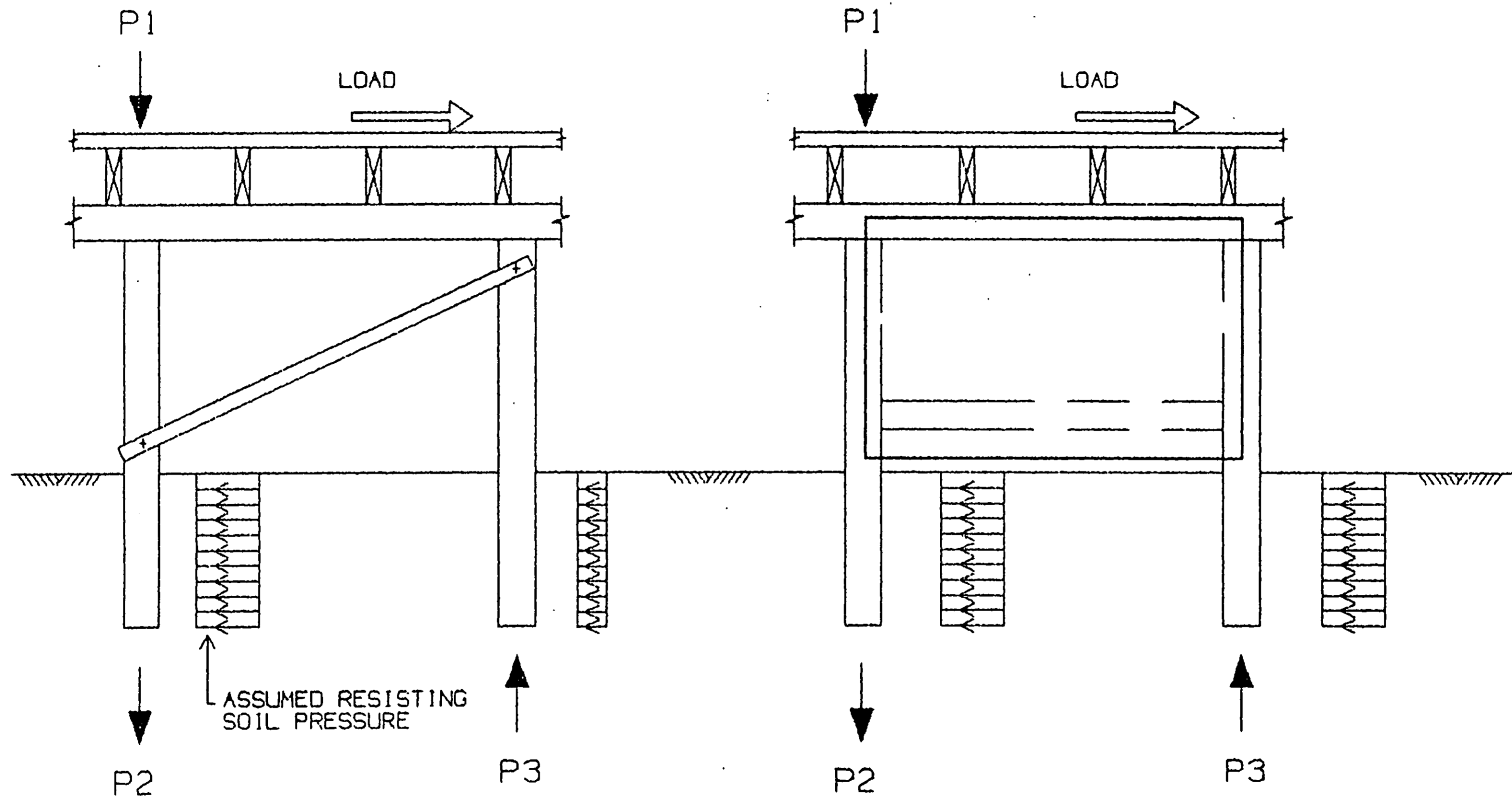
Figure 1 : Seismic or wind forces on a pile



K_+ , K_- Hysteresis loop stiffnesses at Δ_+ , Δ_- respectively (kN/m)
 Δ_+ , Δ_- Ultimate test deflection (mm)
 I Intercept on loop axis as shown at $\Delta=0$ (kN)

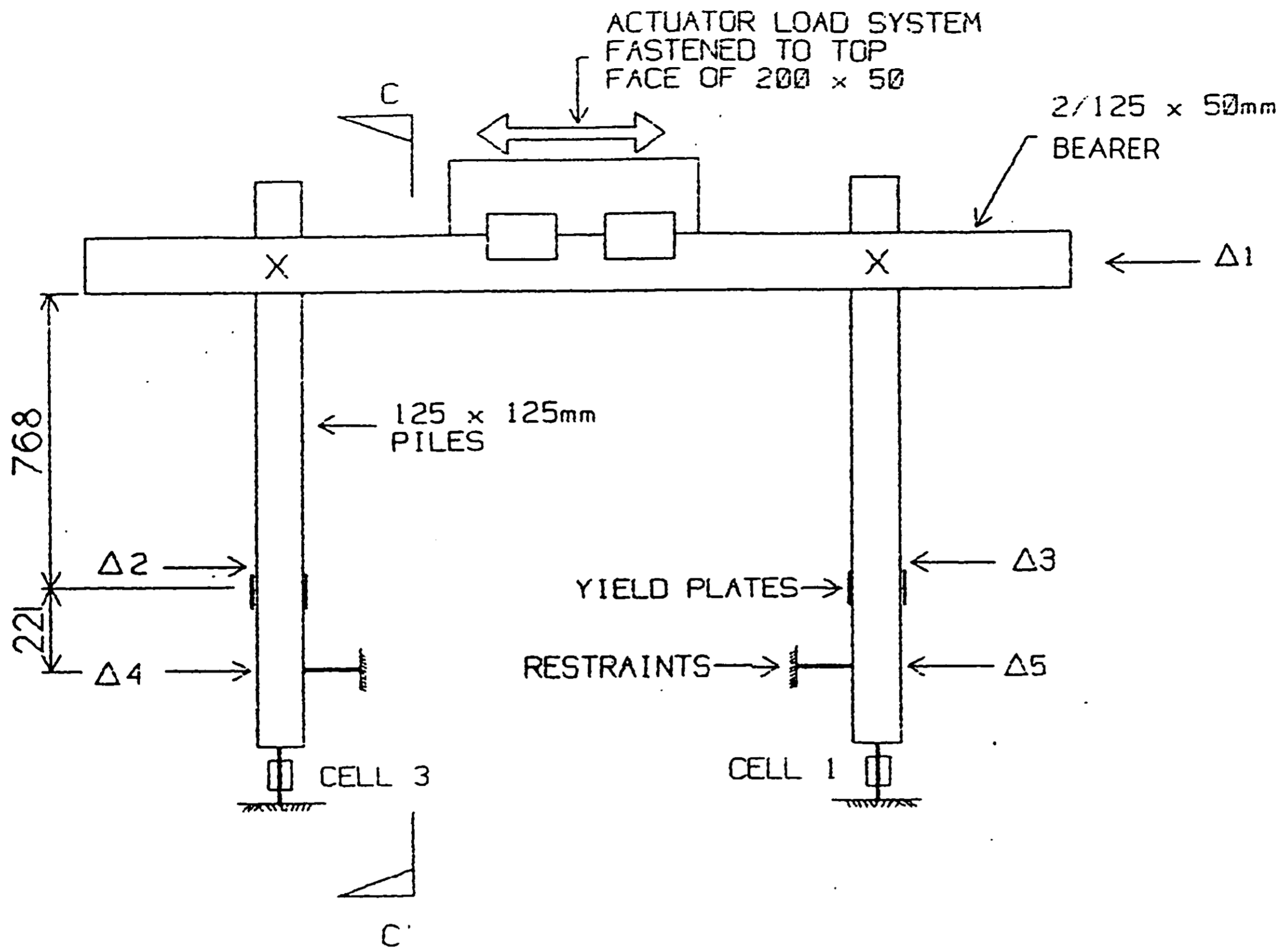
Figure 2 : Data extracted from hysteresis loops for Deans (1992) method

Figure 3 : Soil pressures on piles with restrained pile head

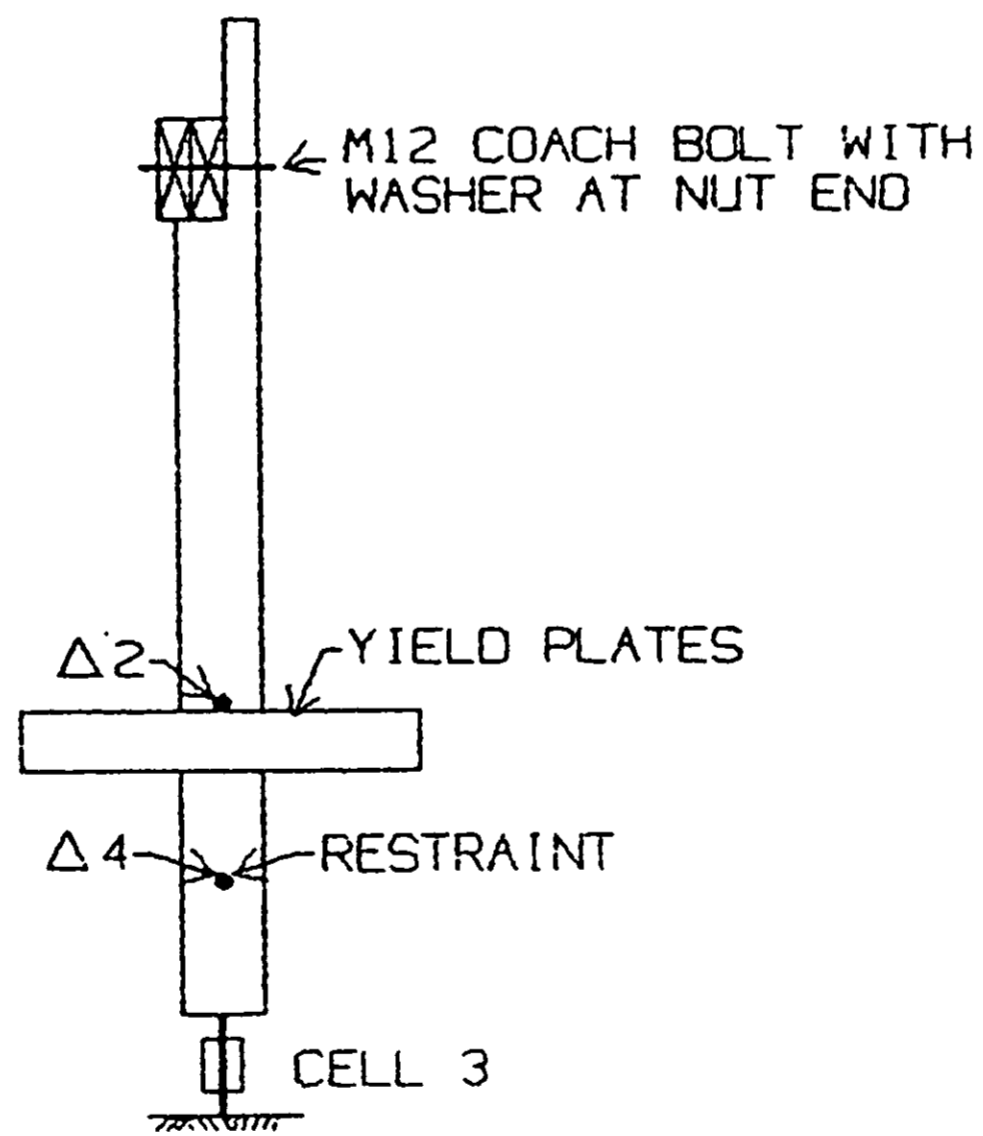


All forces and pressures shown are those due to lateral load alone

- Legend
- P1 Force resisting uplift of floor
 - P2 Force resisting uplift of LHS pile
 - P3 Vertical reaction at bottom of RHS pile



ELEVATION



SECTION C-C'

Figure 4a : Cantilevered pile

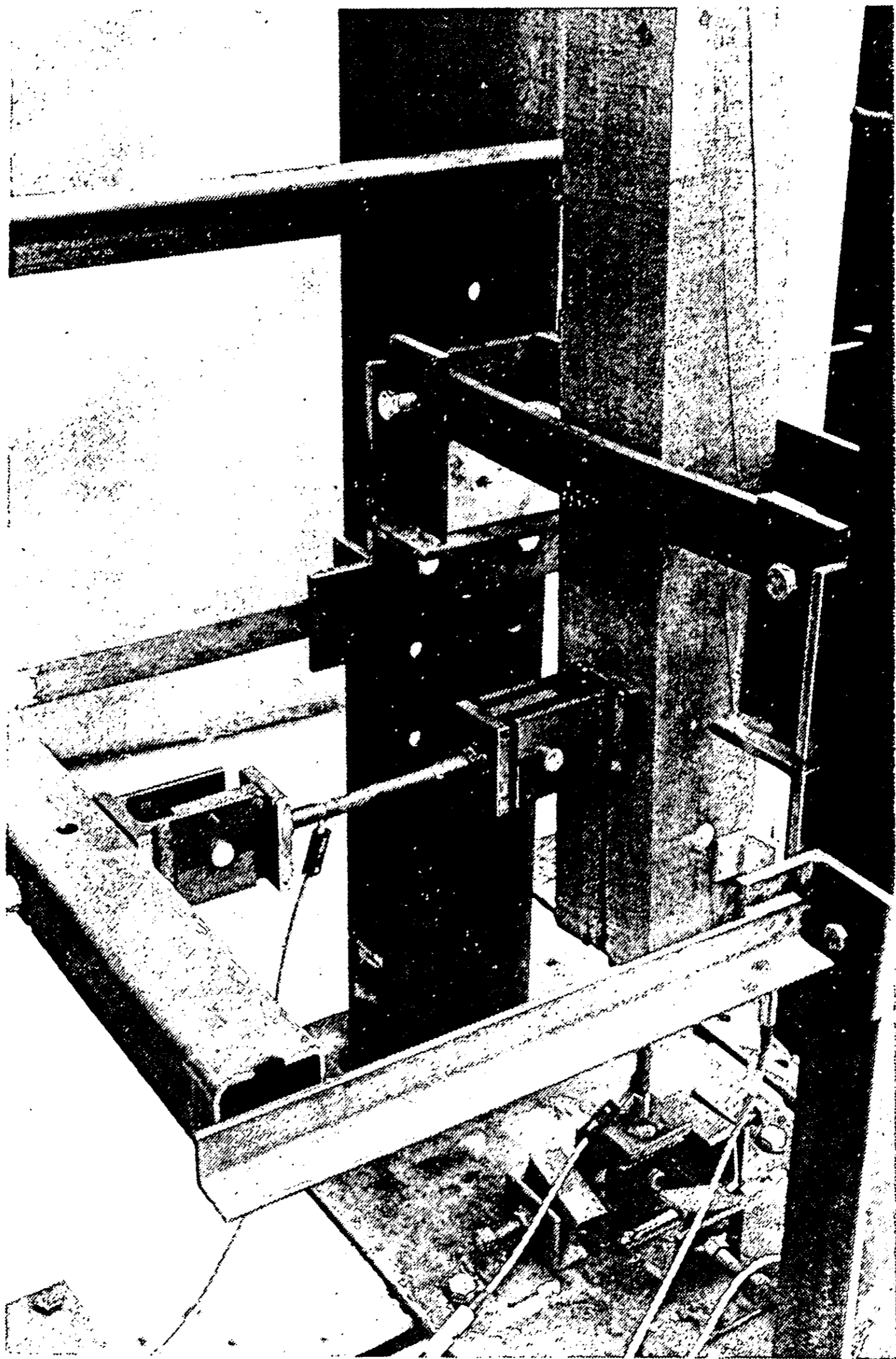
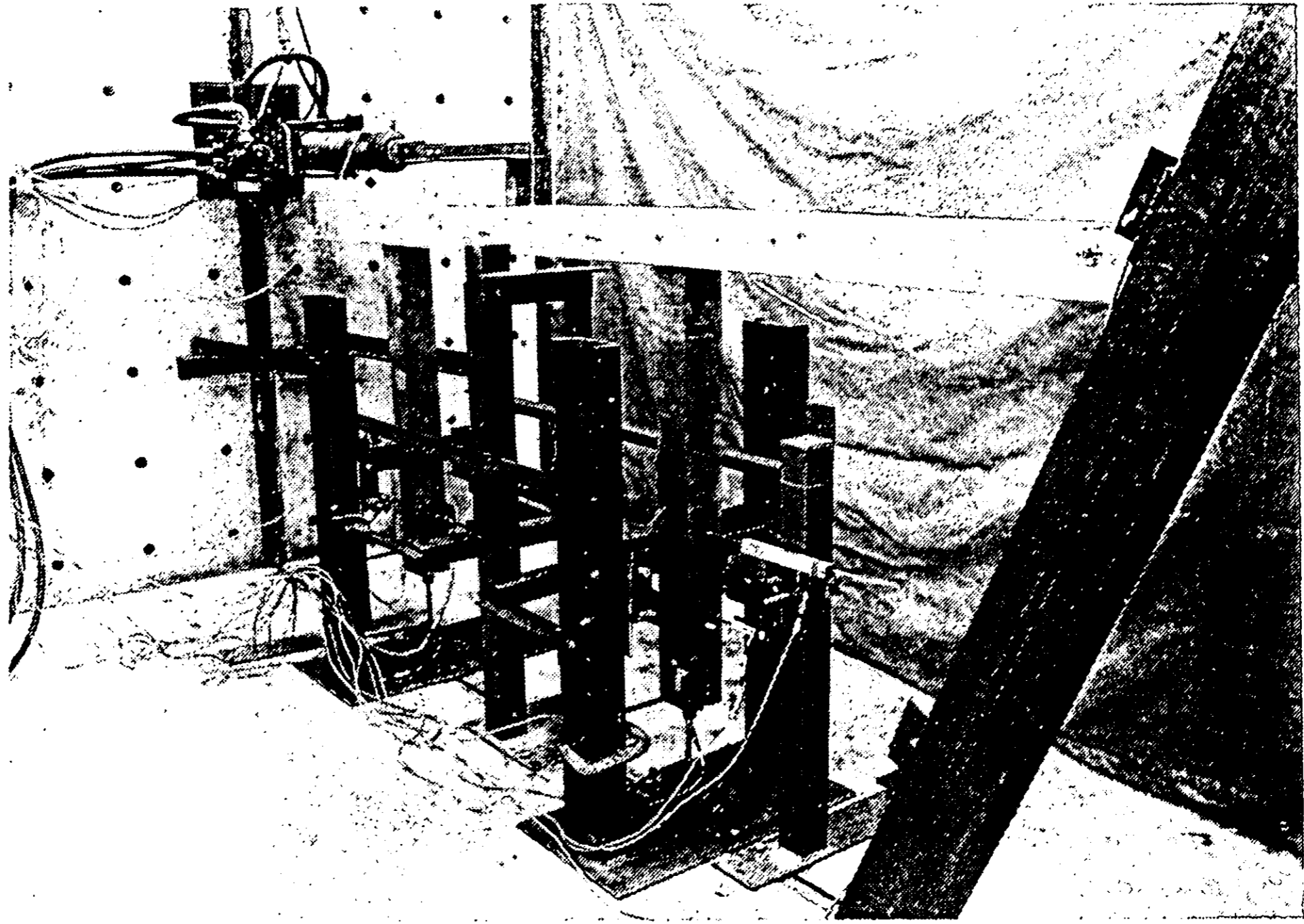


Figure 4b : Cantilevered pile

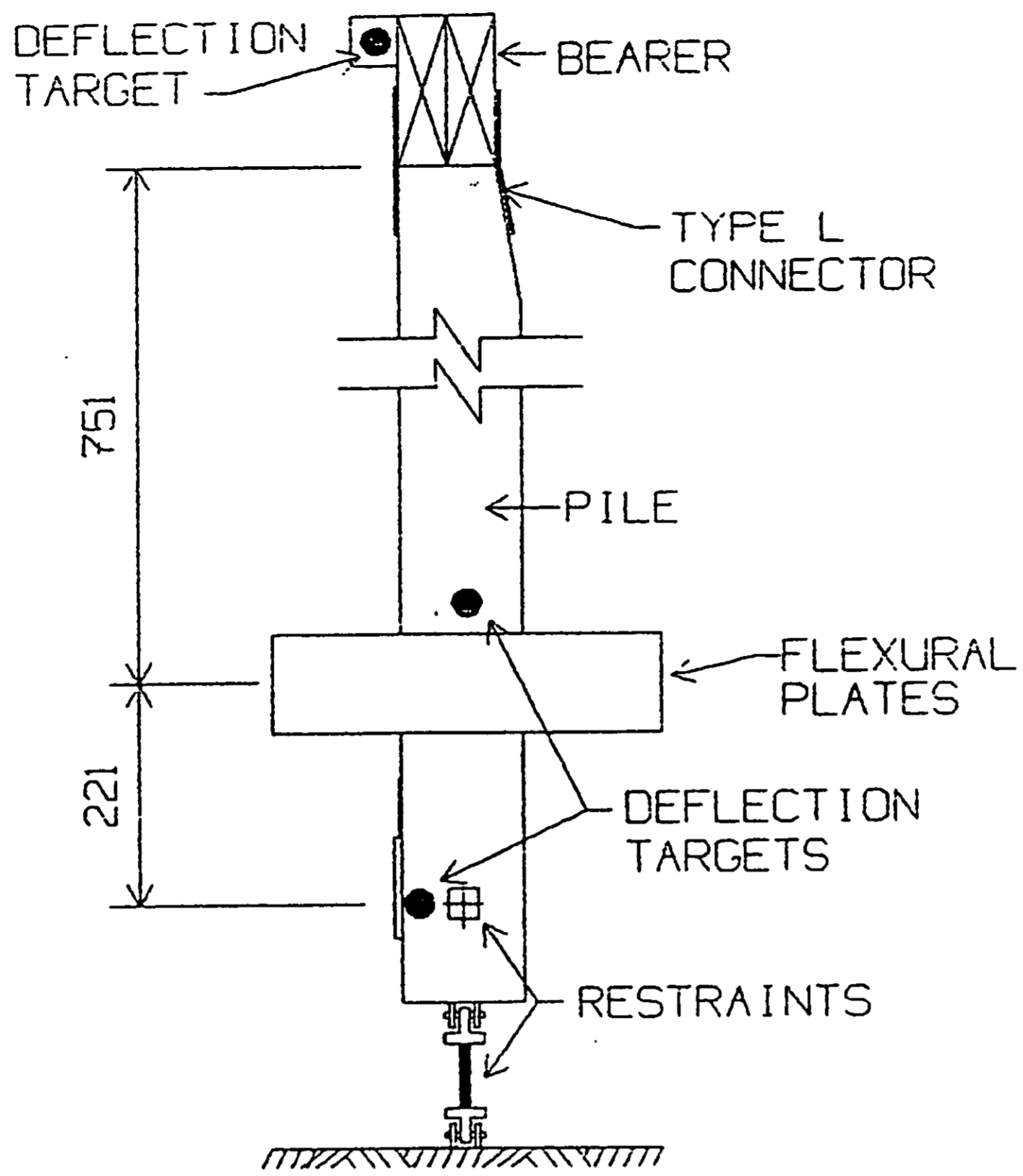


Figure 5a : Replacement of bolt at top of cantilevered pile with type L connectors



Figure 5b : Type L connector

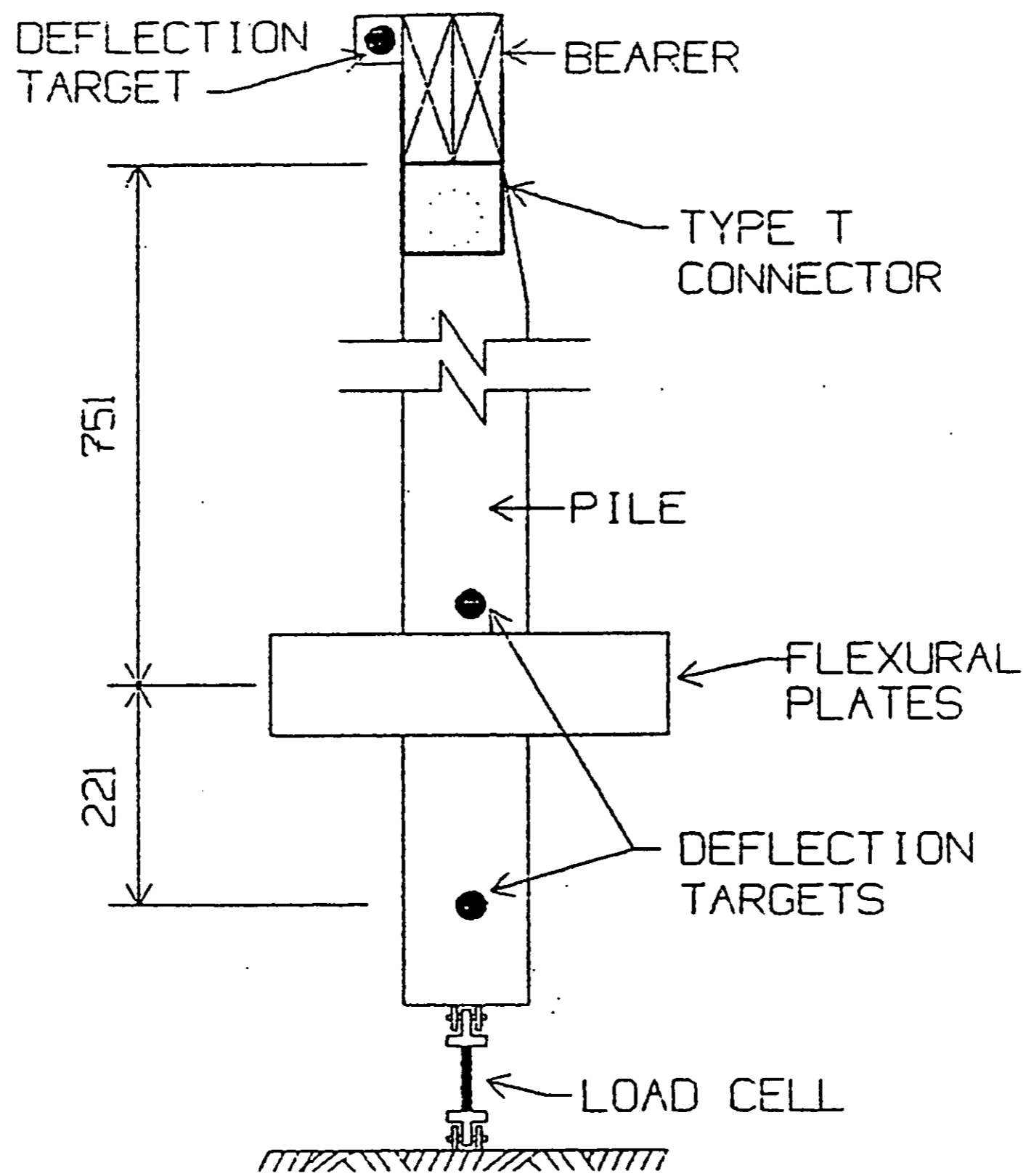


Figure 6a : Replacement of bolt at top of cantilevered pile with type T connectors

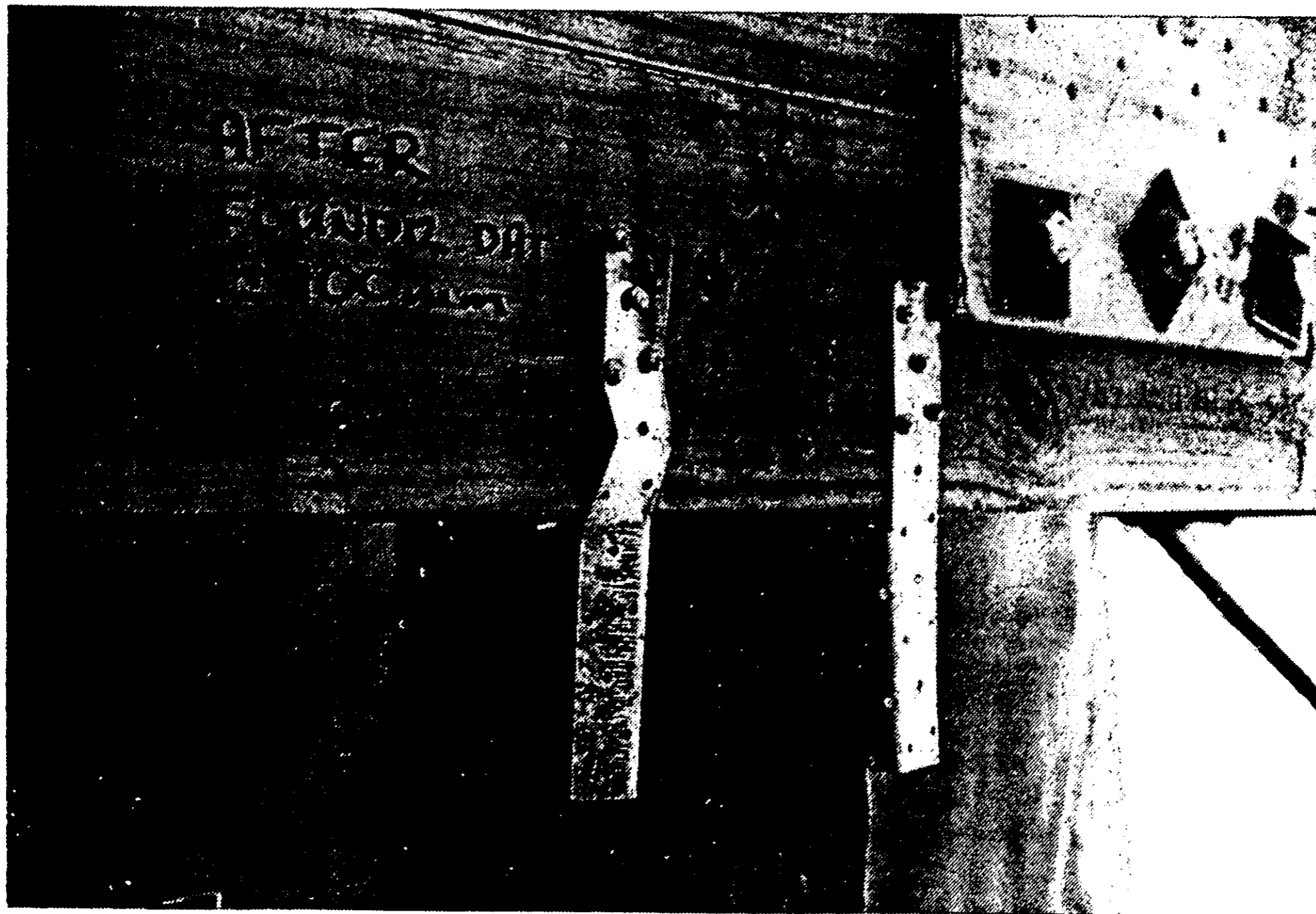


Figure 6b : Type T connector

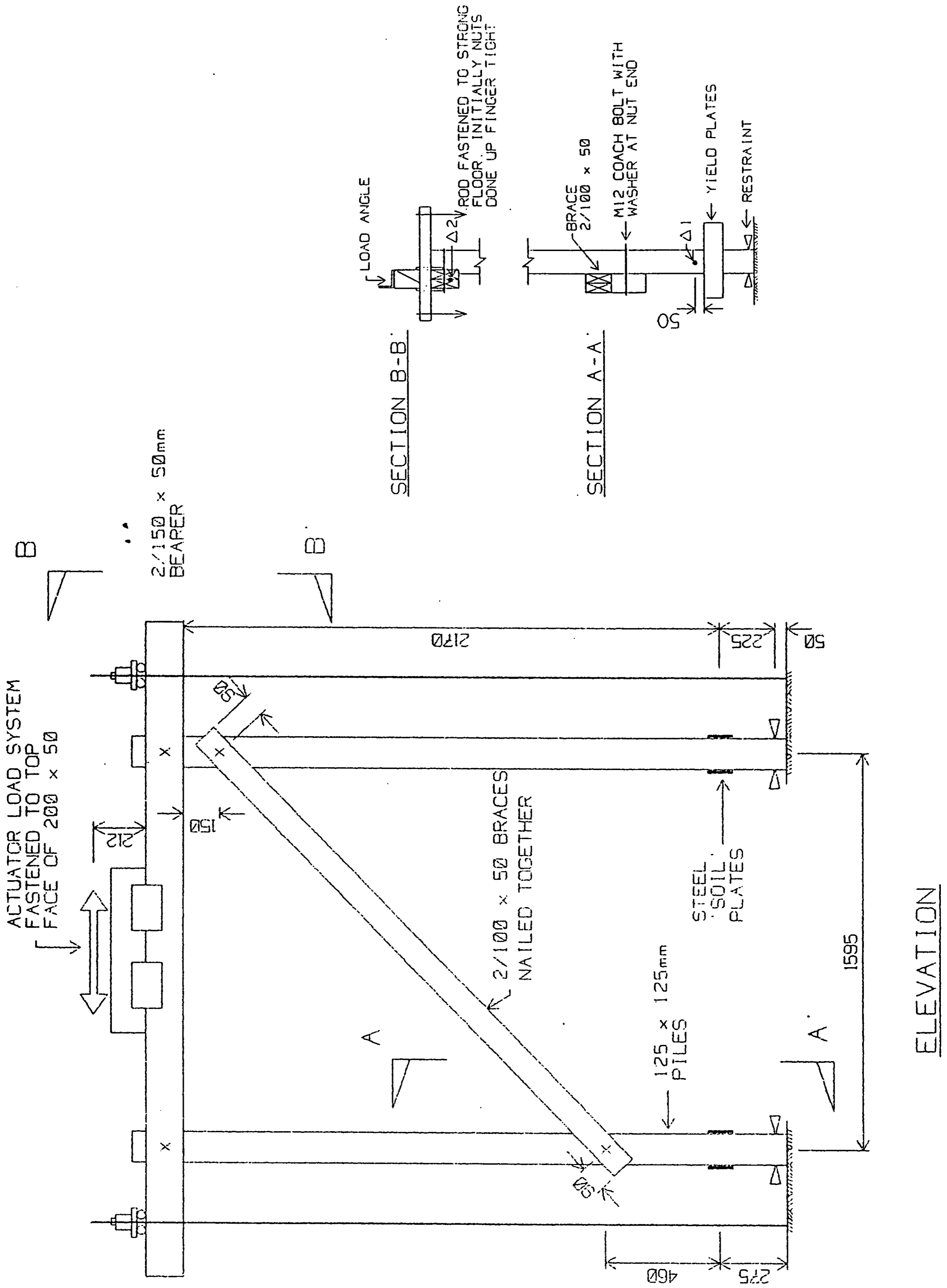
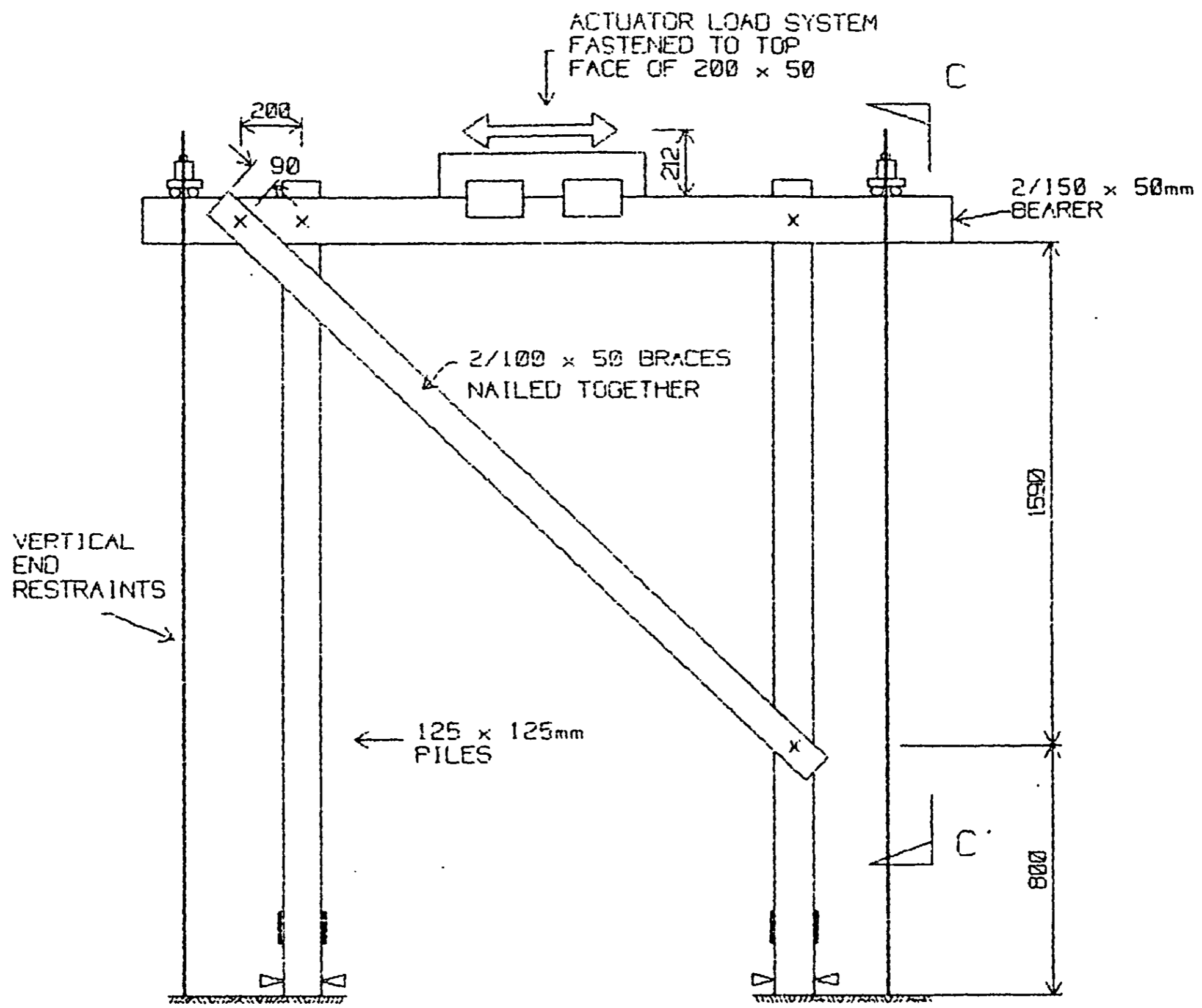
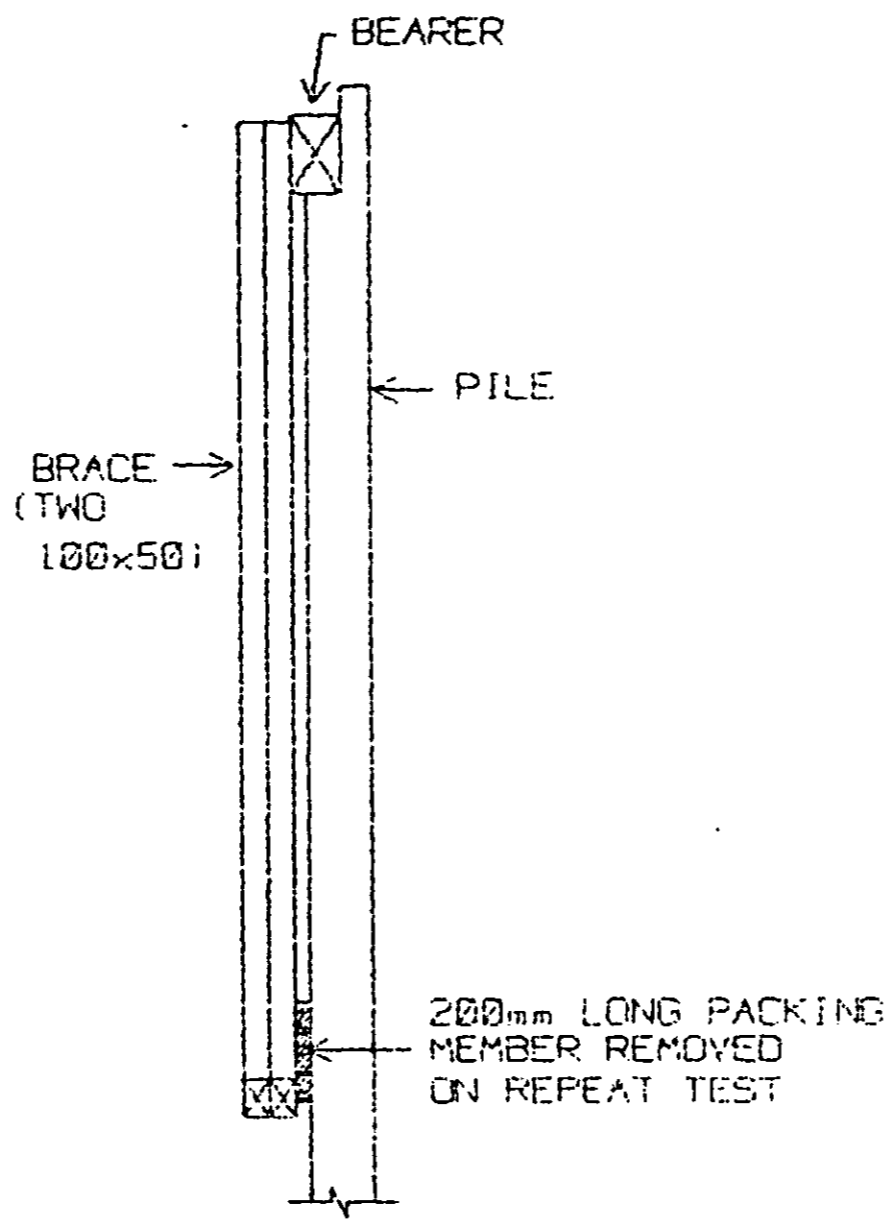


Figure 7 : Test set-up braced piles : Brace from pile to pile



ELEVATION



SECTION C-C'

Figure 8a : Test set-up braced piles : Brace from pile to bearer

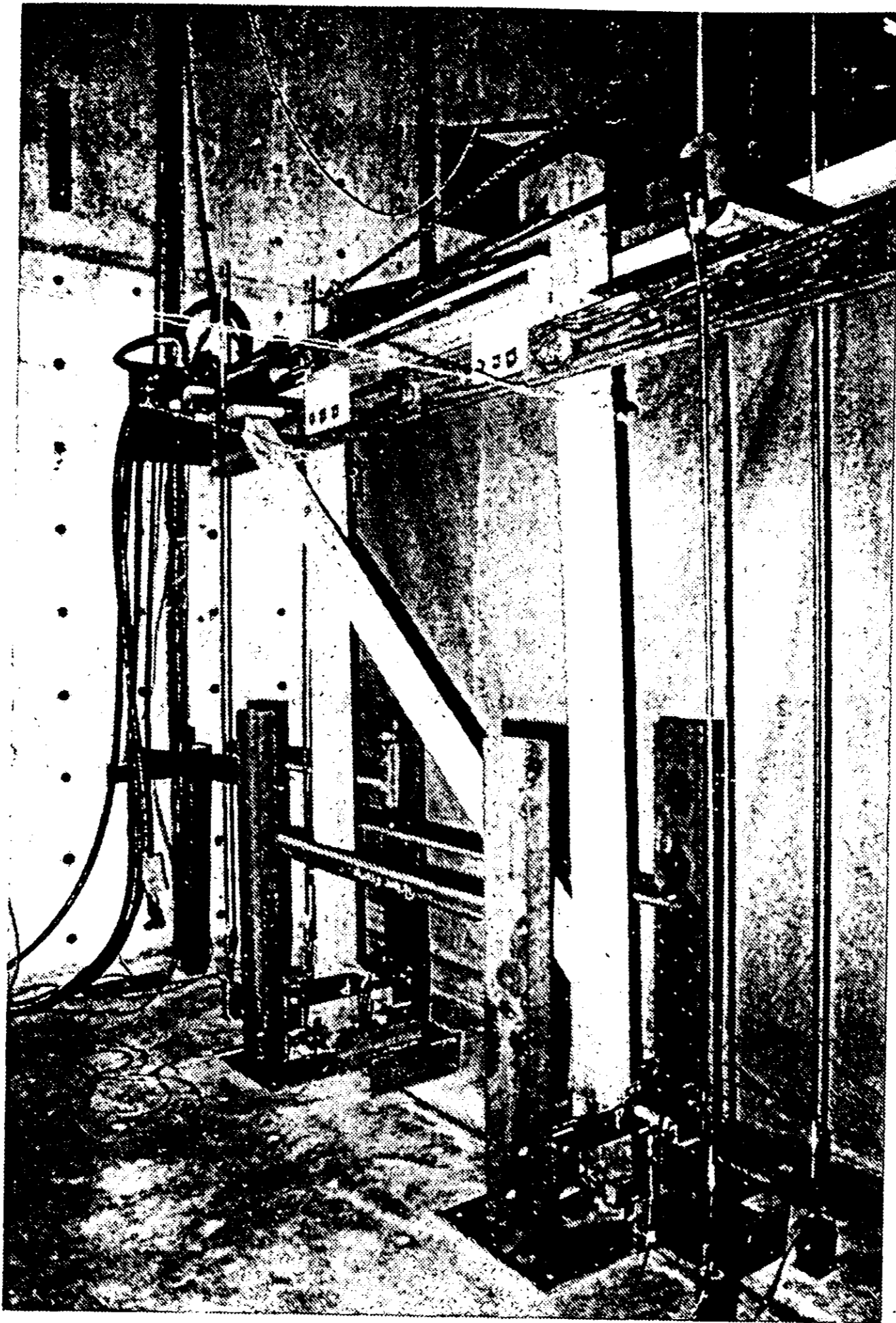


Figure 8b : Test set-up

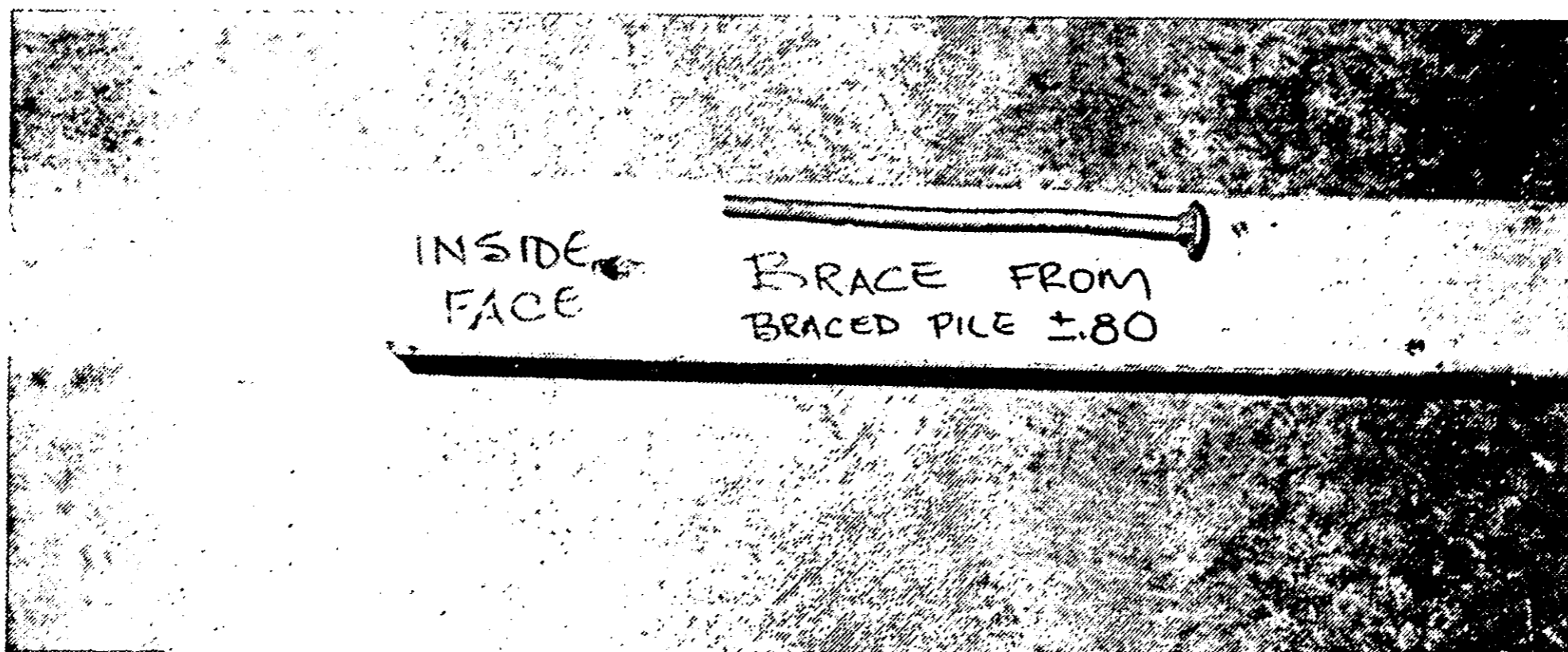


Figure 8c : Damage to coachbolt and brace

PLATES ONLY CANTILEVER PILE FOUND2-4

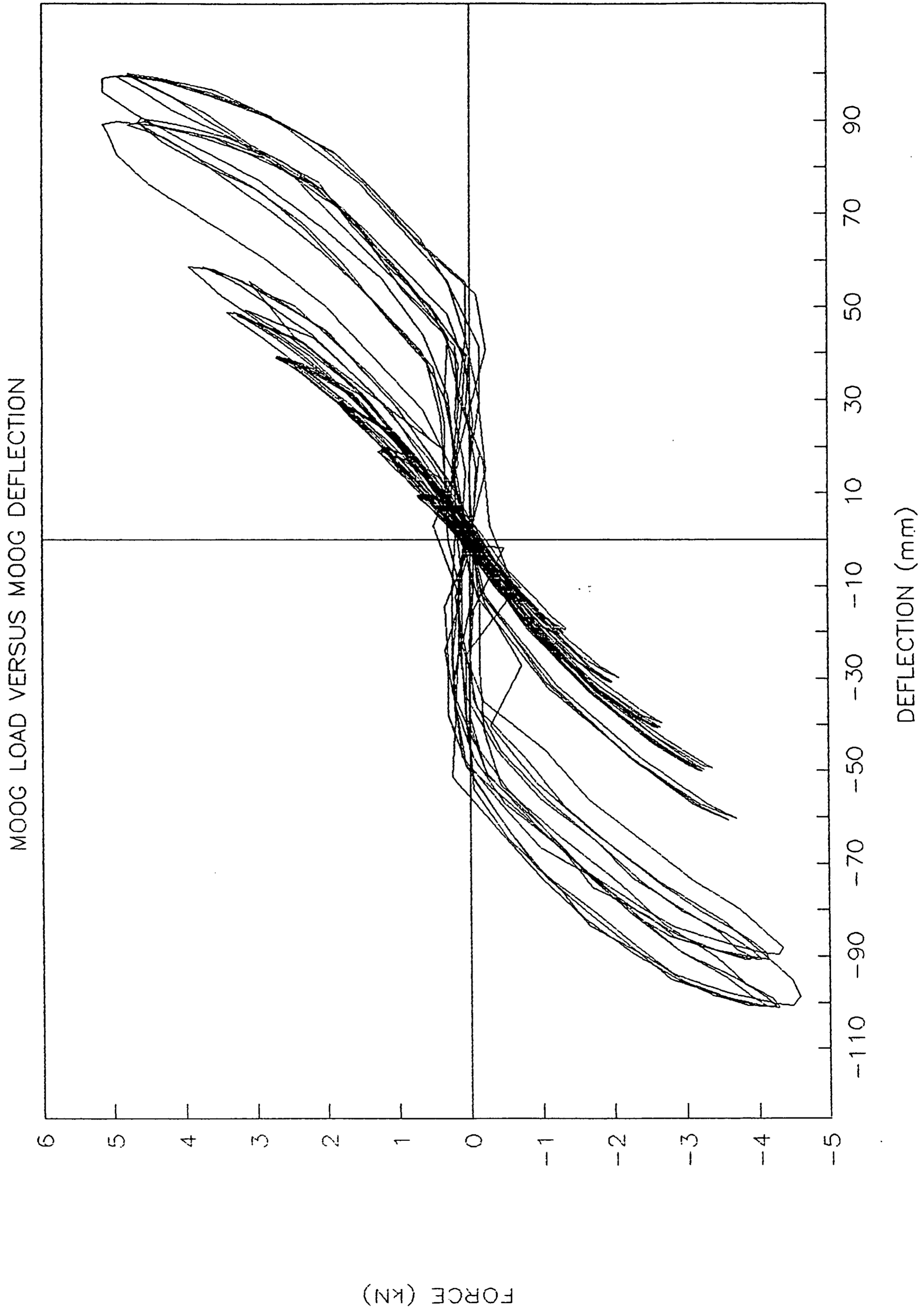


Figure 9 : Hysteresis loops for short cantilevered piles with 60x12 plates only

TYPE L CANTILEVER PILES: (FIRST RUN)

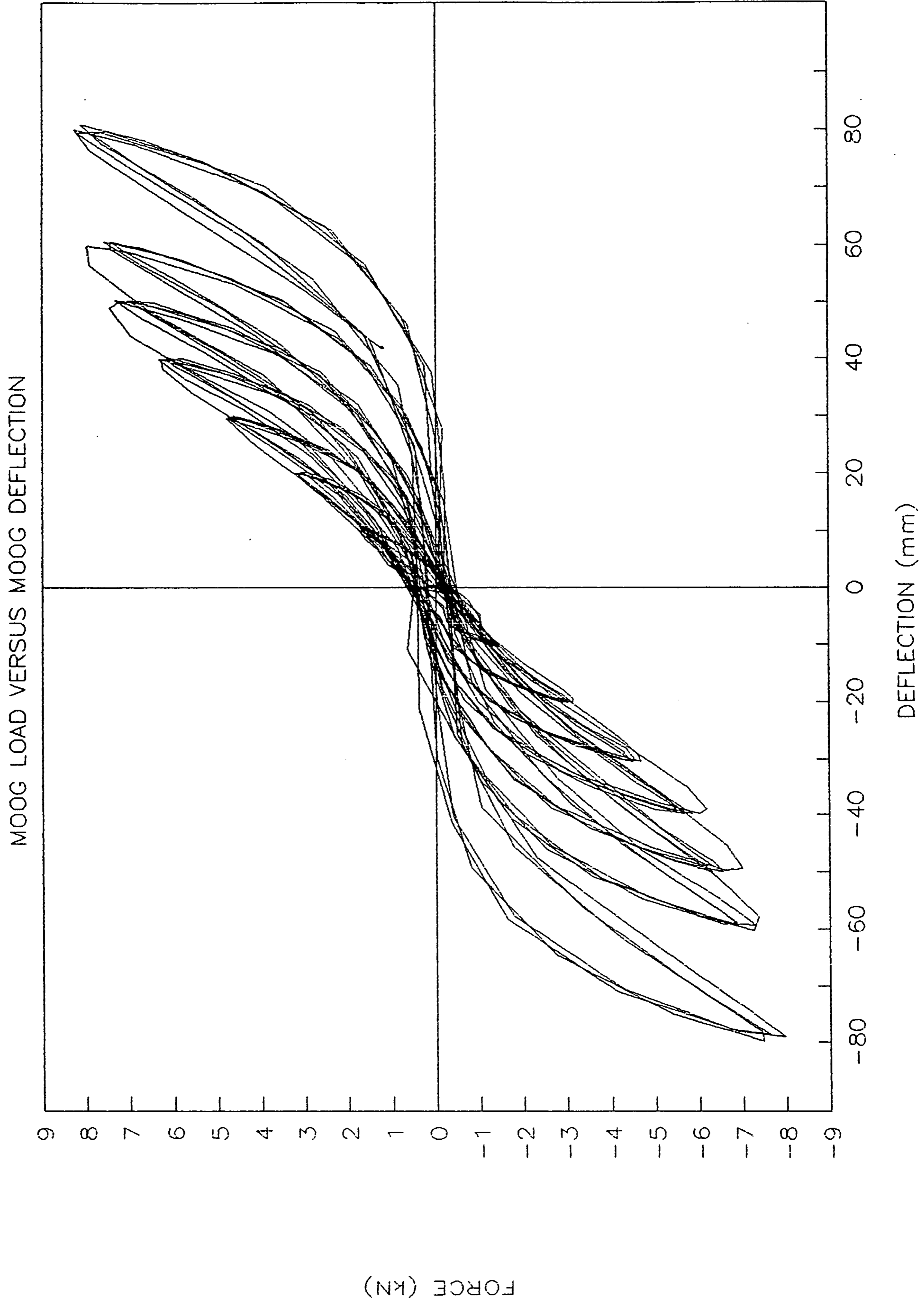


Figure 10 : Hysteresis loops for short cantilevered piles with 60x12 plates and type L top connection

TYPE L CANTERLEVER PILES (SECOND RUN)

MOOG LOAD VERSUS MOOG DEFLECTION

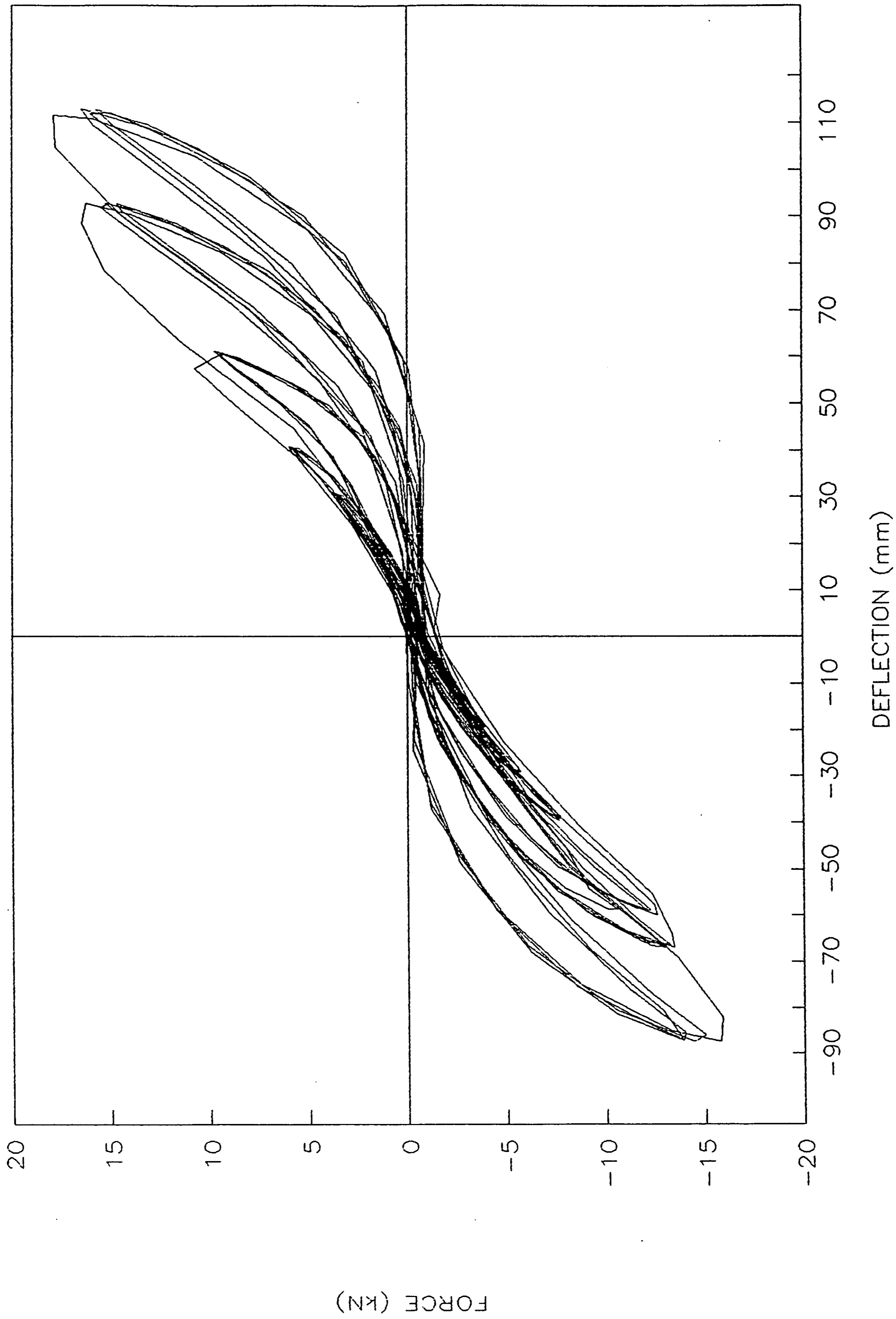


Figure 11 : Hysteresis loops for short cantilevered pile with 200x12 soil plates and type L top connection

TYPE T CANTERLEVER PILES (FIRST RUN)

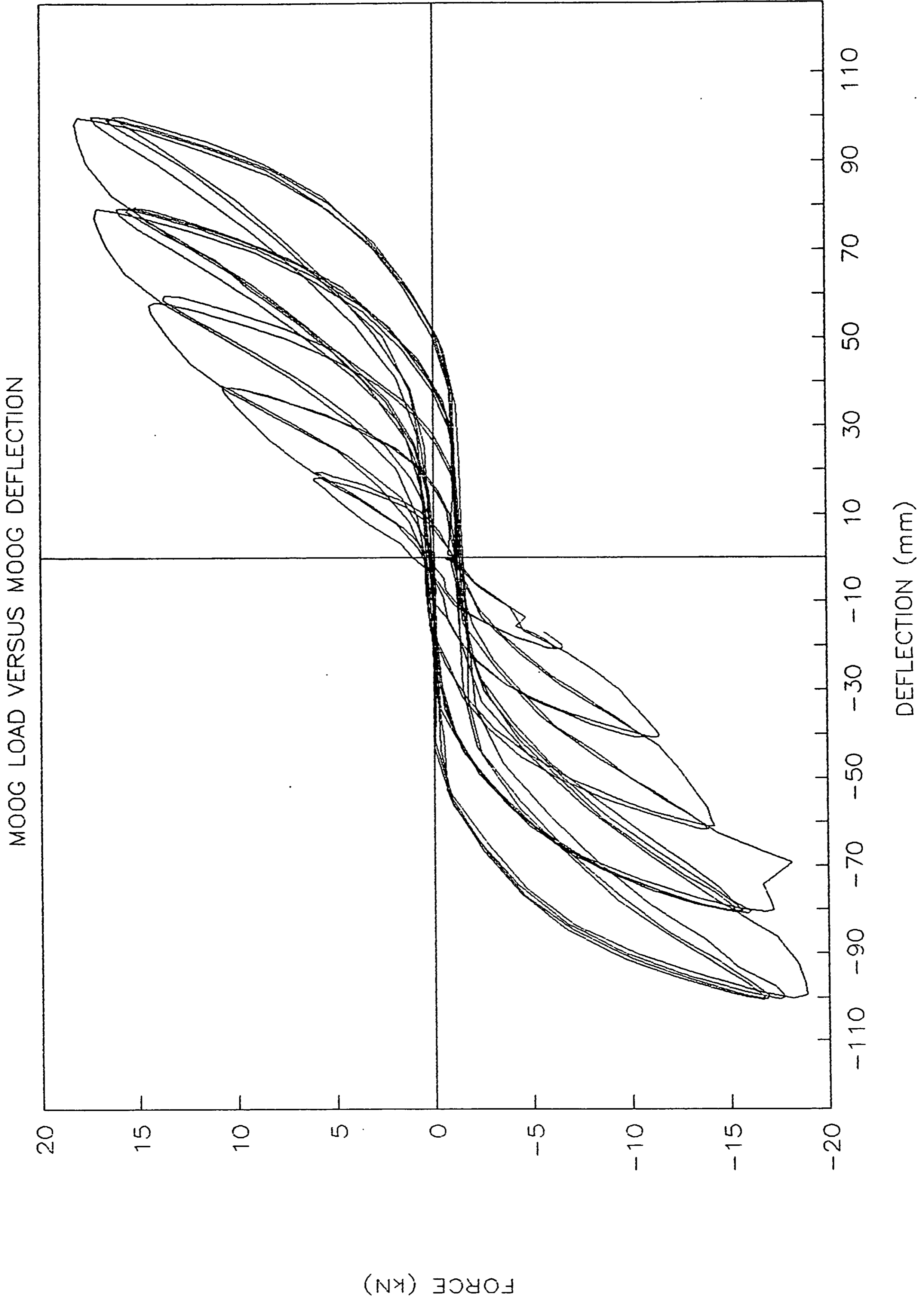


Figure 12 : Hysteresis loops for short cantilevered pile with 200x12 soil plates and type T top connection

TYPE T CANTERLEVER PILES (SECOND RUN)

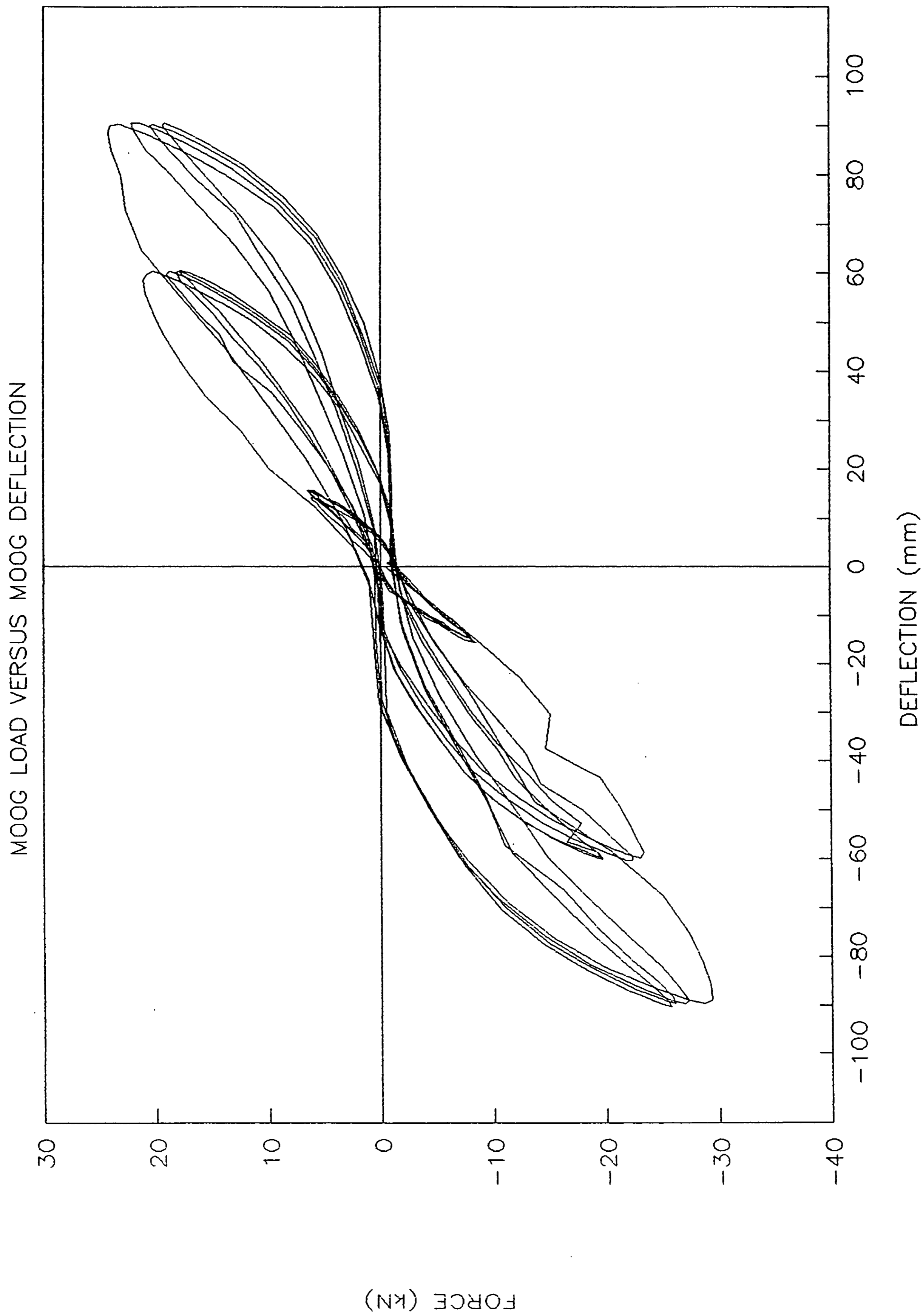


Figure 13 : Hysteresis loops for short cantilevered pile with double plates and type T top connection

NO BRACE AND 12 MM PLATES (FIRST RUN)

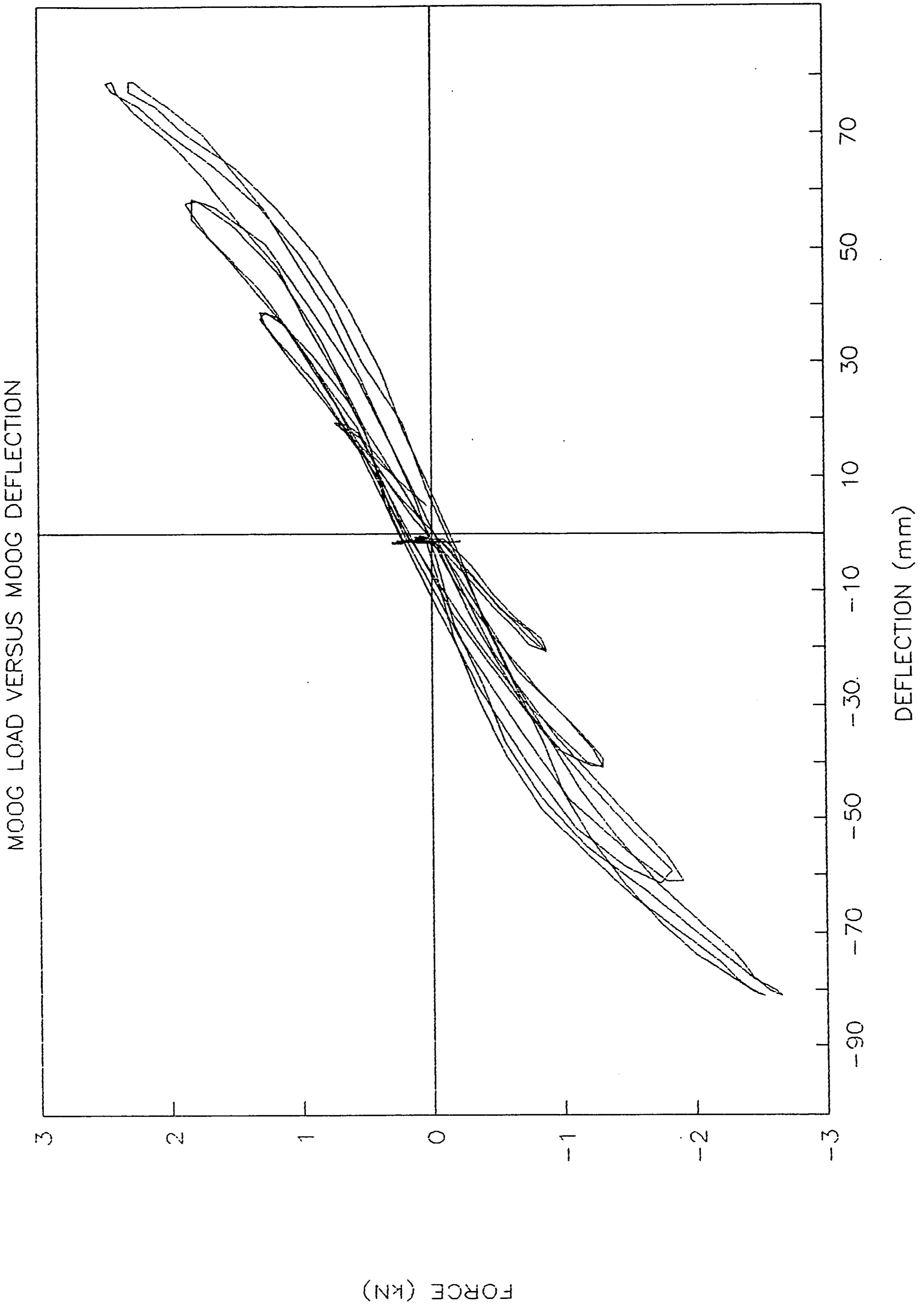


Figure 14 : Hysteresis loops for pile without brace and plates only

SOLID BRACE FROM PILE TO PILE

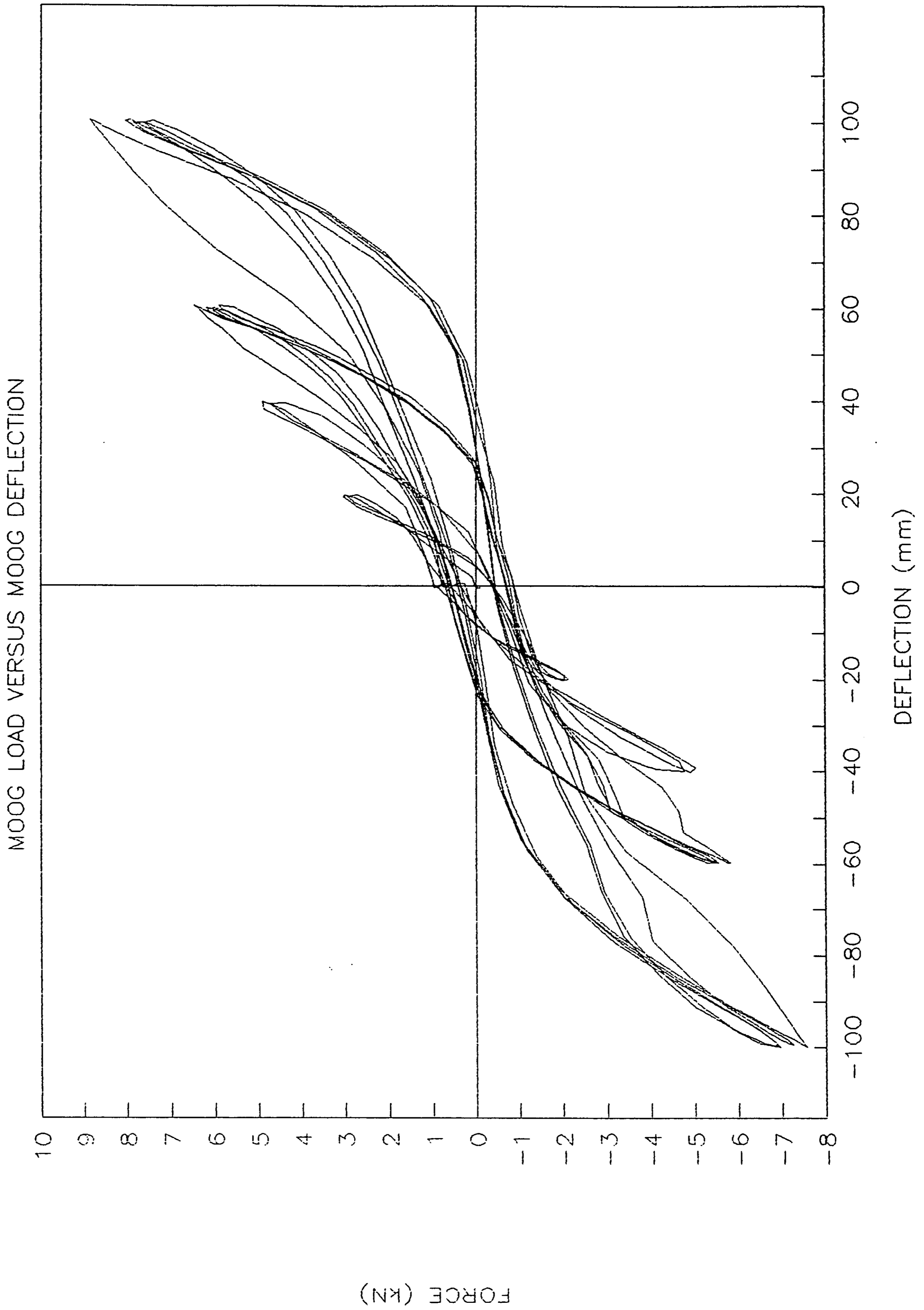


Figure 15 : Hysteresis loops for braced pile with brace from pile to pile

SOLID BRACE PILE TO BEARER (BLOCKING)

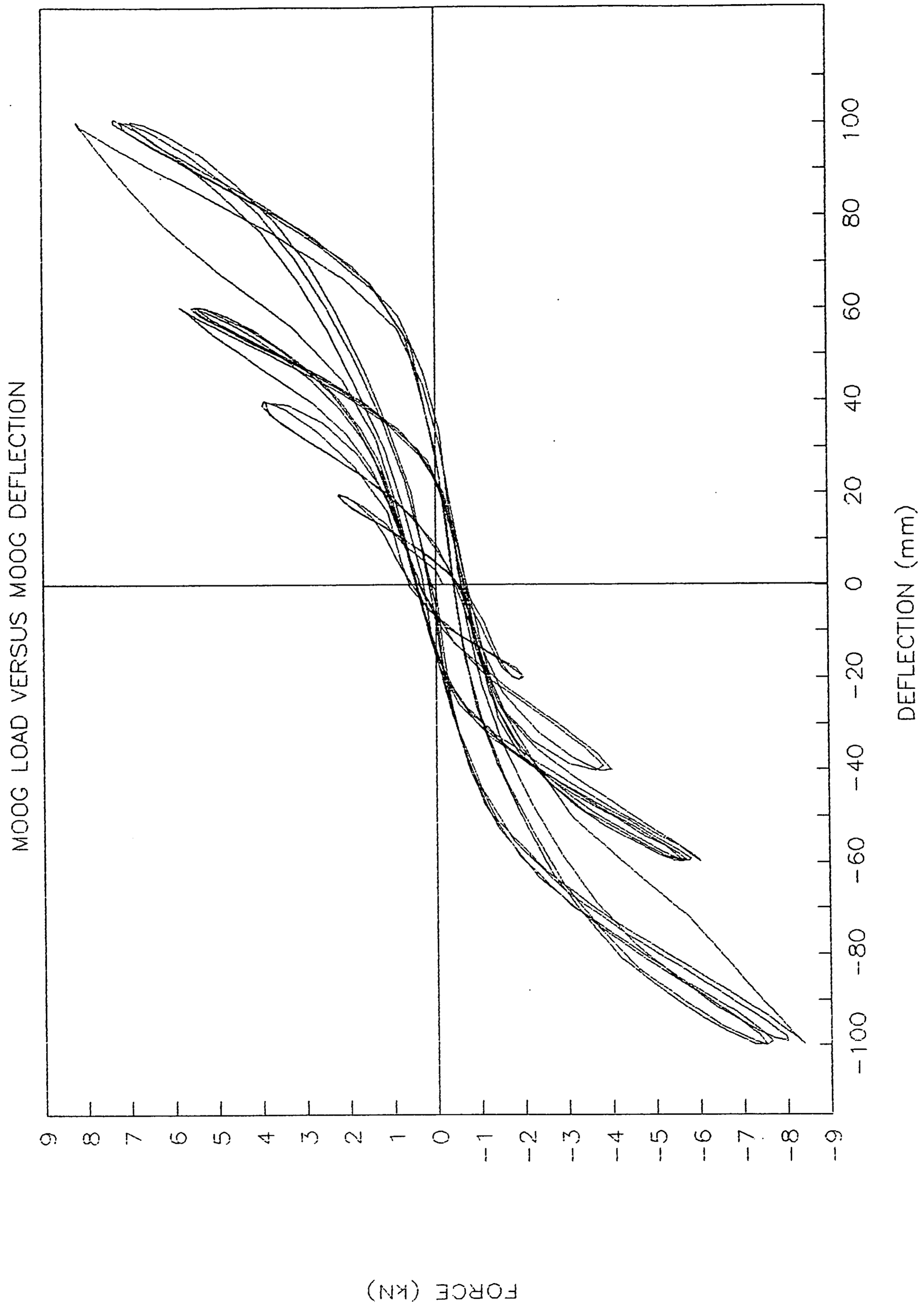


Figure 16 : Hysteresis loops for braced pile with brace from pile to bearer (with packer)

SOLID BRACE PILE/BEARER (NO BLOCKING)

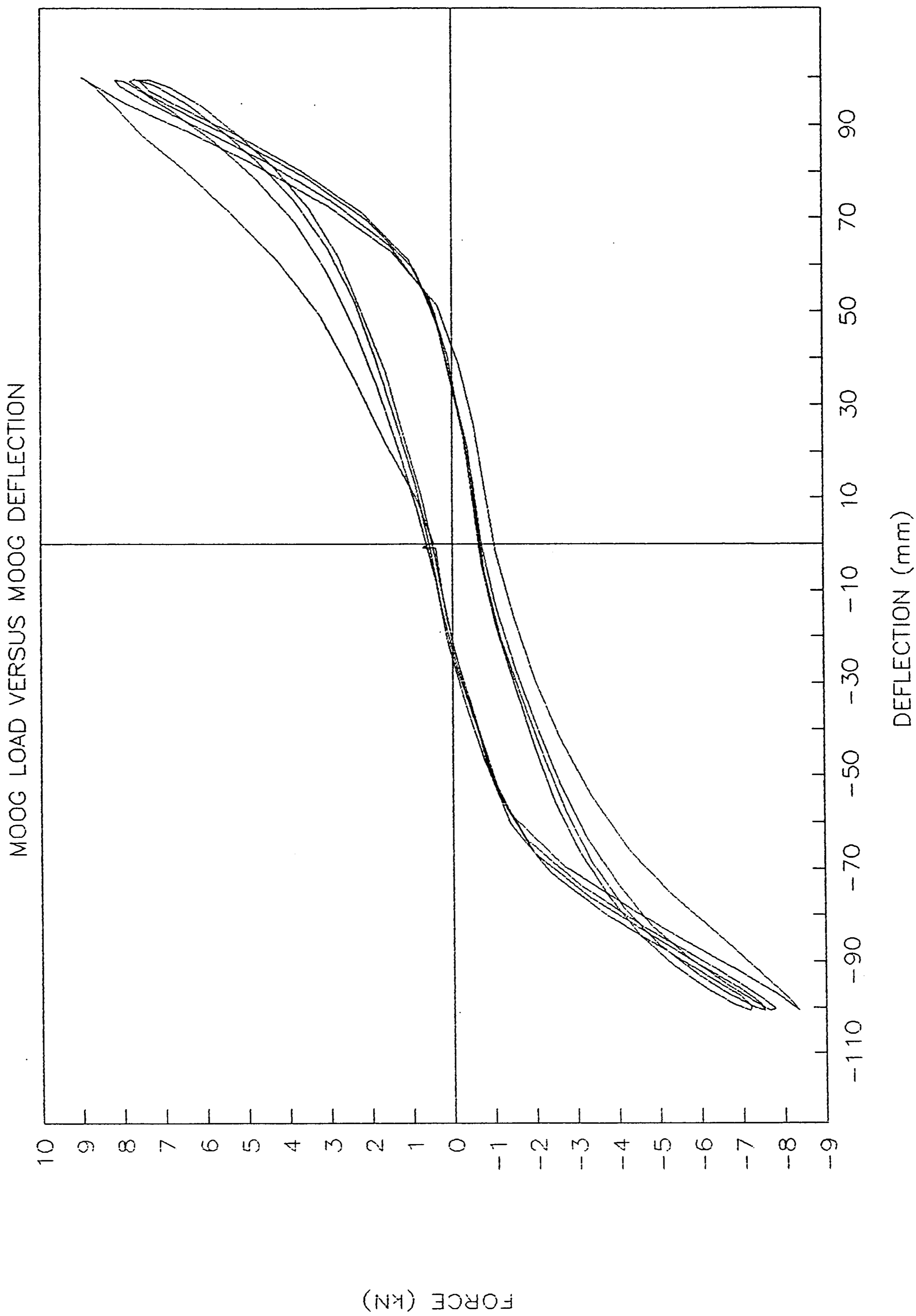


Figure 17 : Hysteresis loops for braced pile with brace from pile to bearer (no packer)

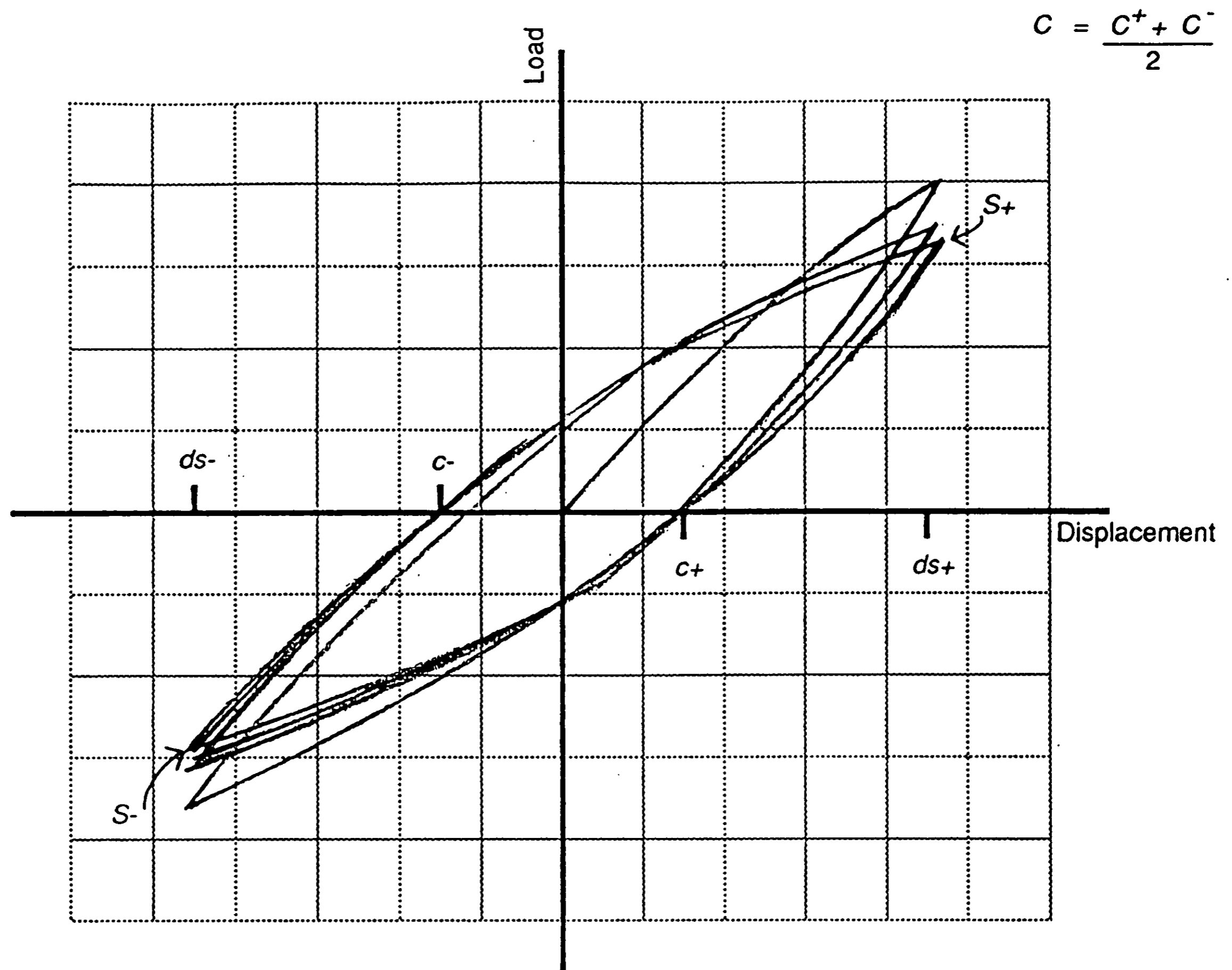


Figure A.1 : Measurements during displacement to ds

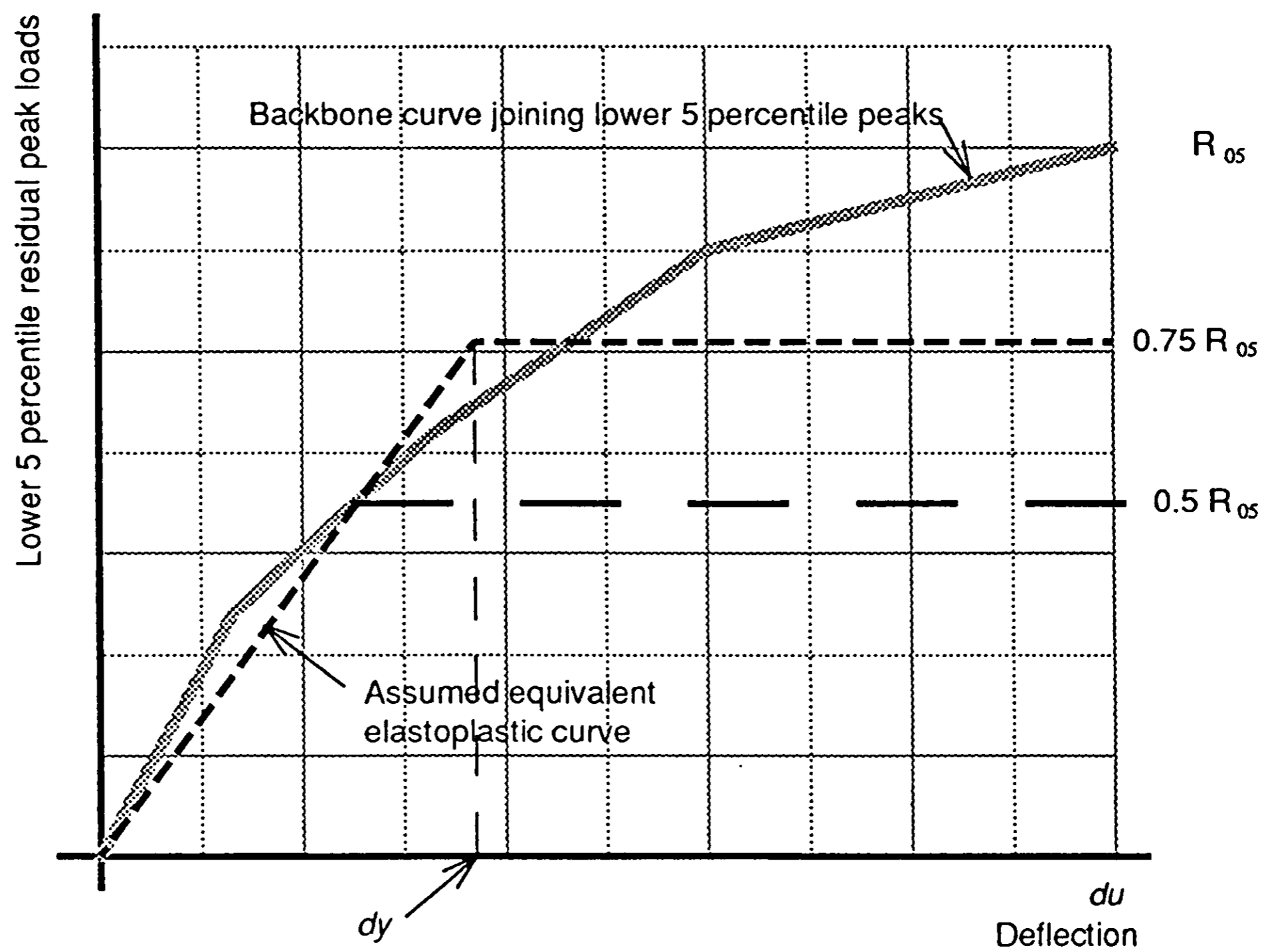
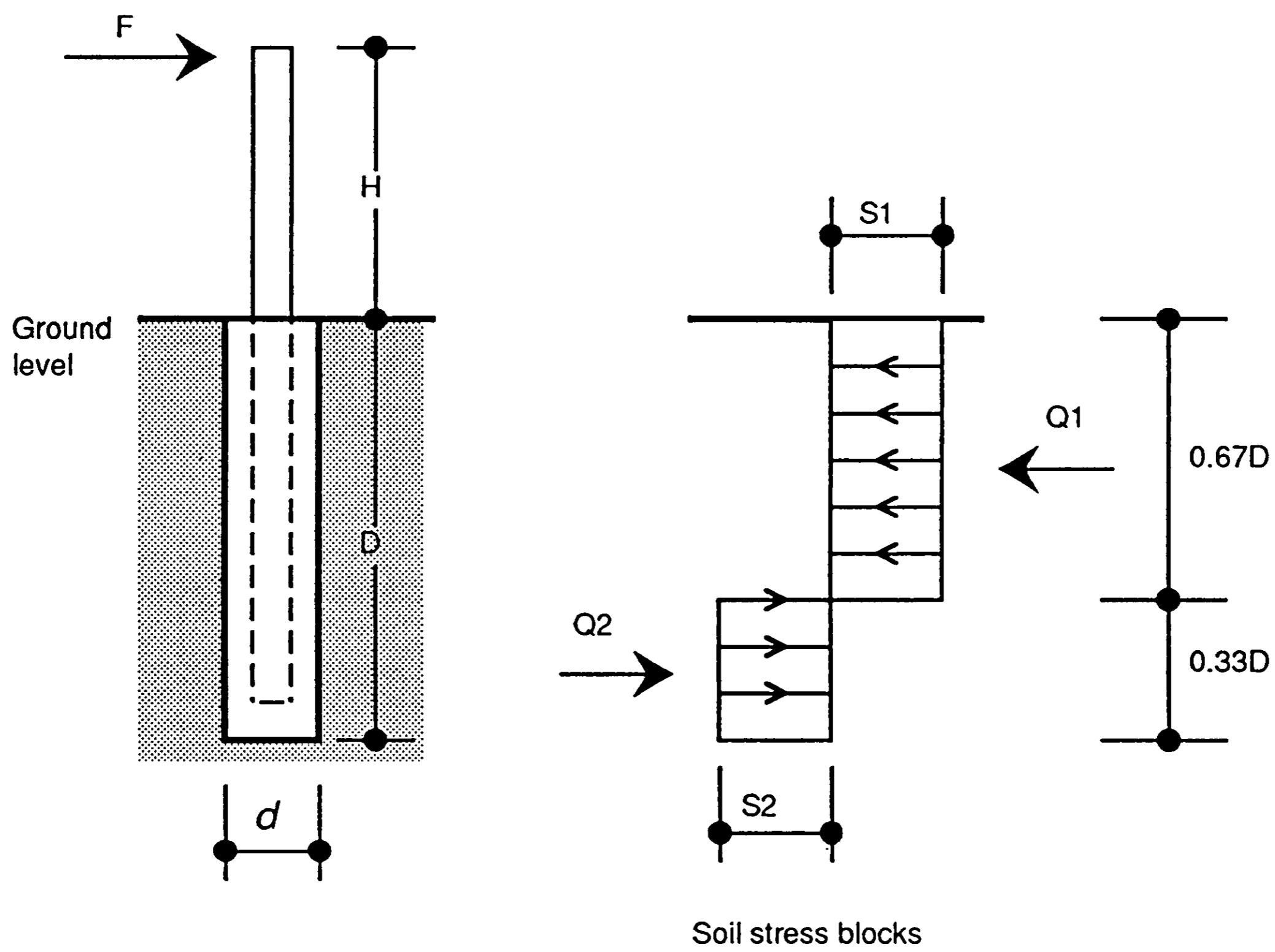


Figure A.2 : Equivalent elastoplastic yield load and ductility

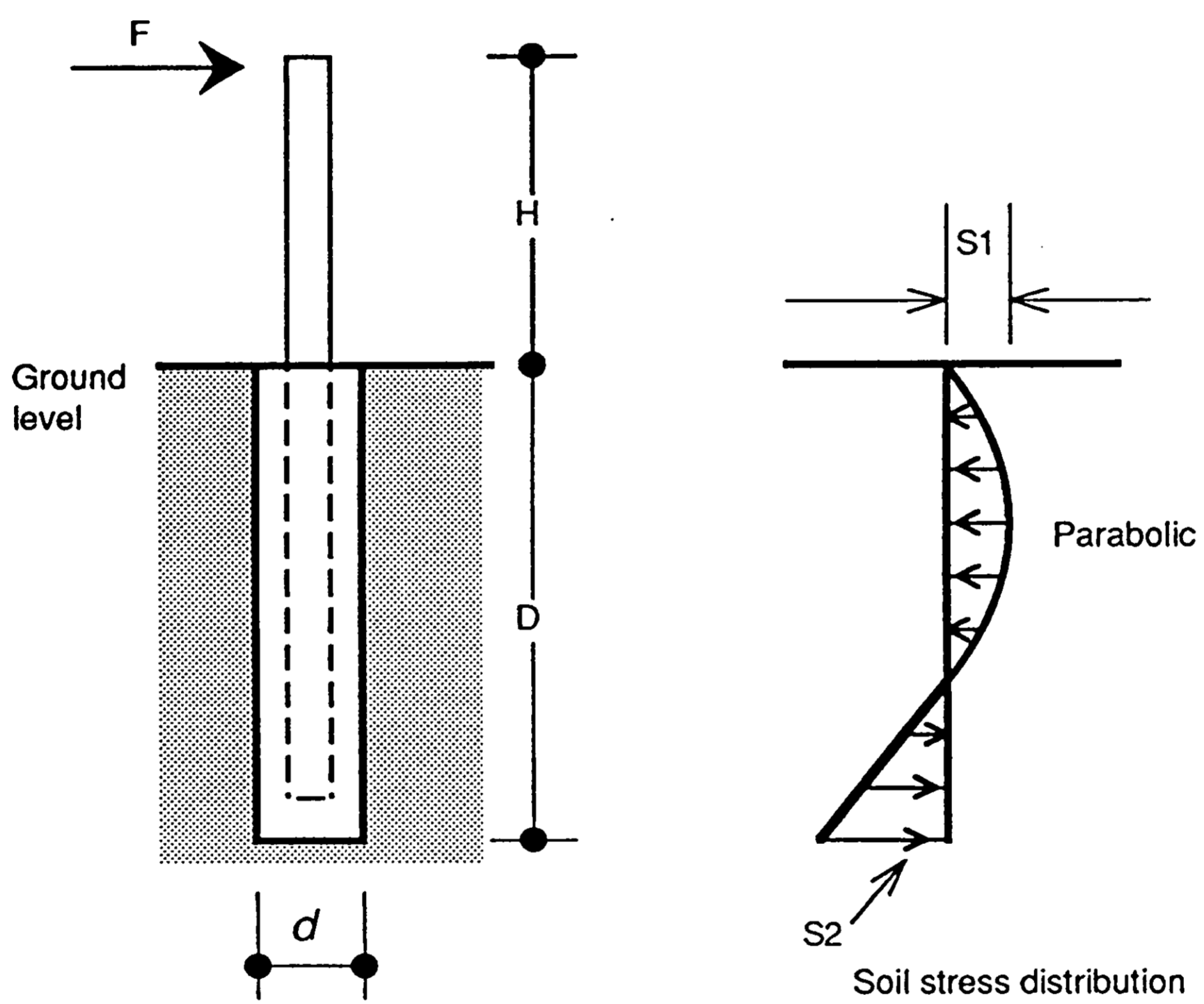


From statics:

$$S_1 = \frac{F(H + 0.83D)}{0.33D^2d} \quad ; \quad Q_1 = \frac{F(H + 0.83D)}{0.5D}$$

$$S_2 = \frac{F(H + 0.33D)}{0.17D^2d} \quad ; \quad Q_2 = \frac{F(H + 0.33D)}{0.5D}$$

Figure B.1 : Short pile solution for rectangular soil stress blocks



$$S_1 = \frac{F(H + 8D)}{9} \frac{81}{20D^2 d}$$

$$S_2 = \frac{F(H + D)}{3} \frac{54}{5D^2 d}$$

Figure B.2 : Short pile solution for parabolic soil stress blocks

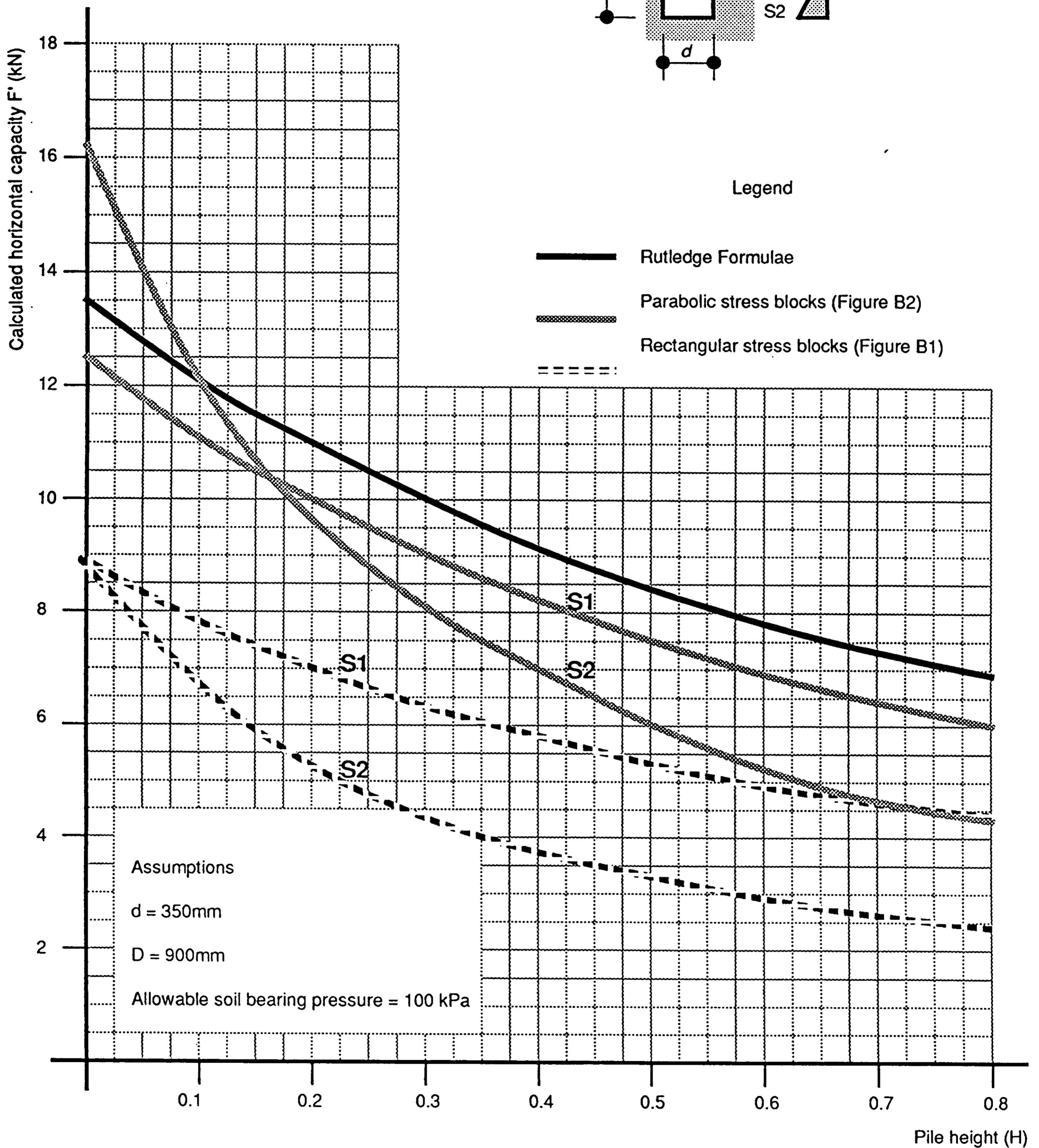
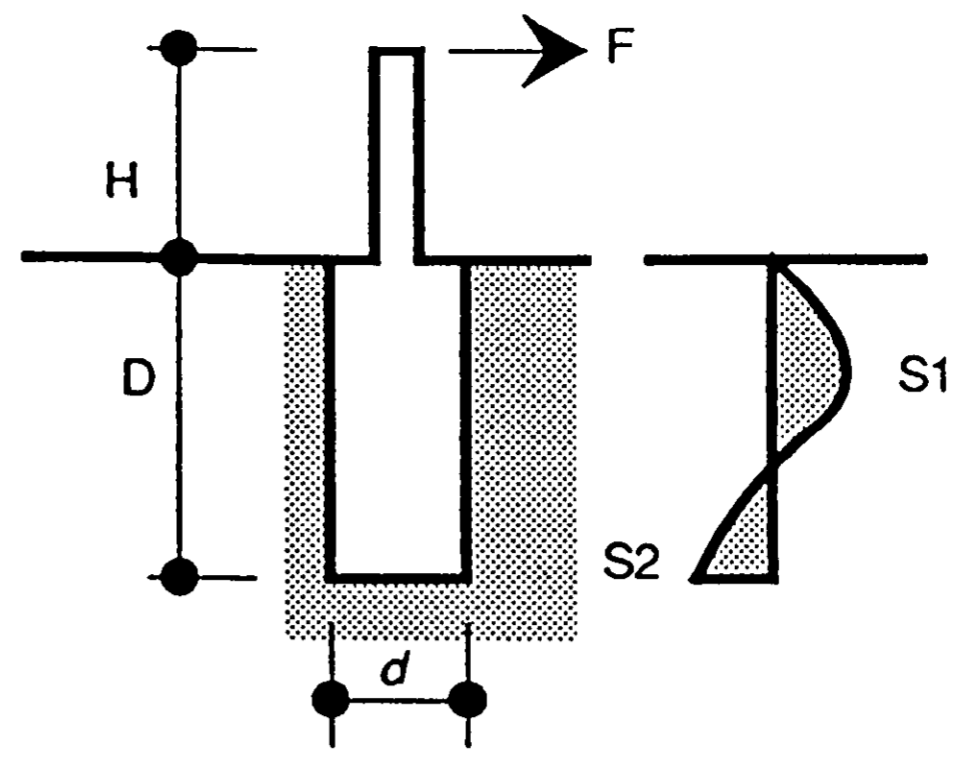
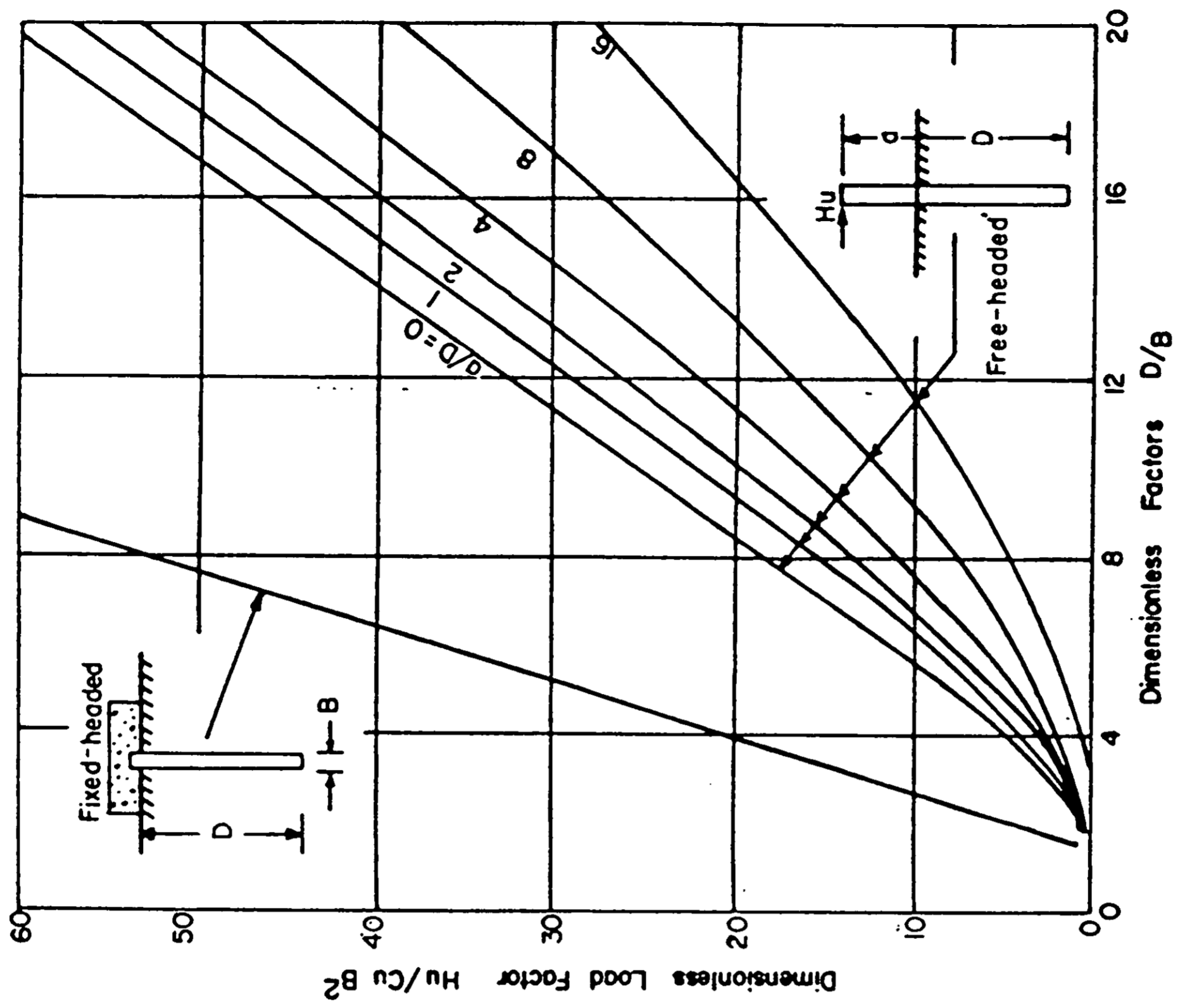
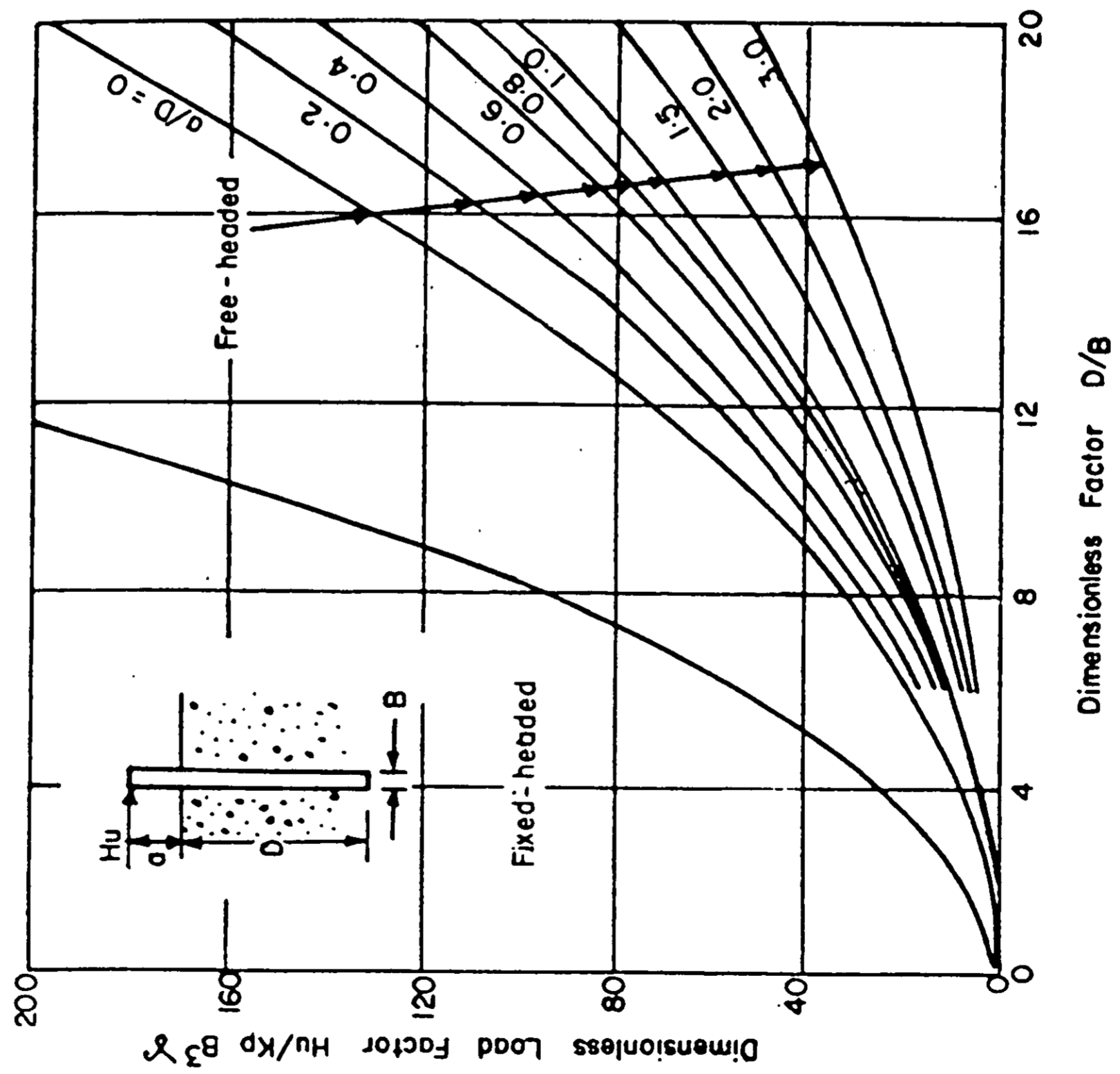


Figure B.3 : Pile lateral load capacity using various methods

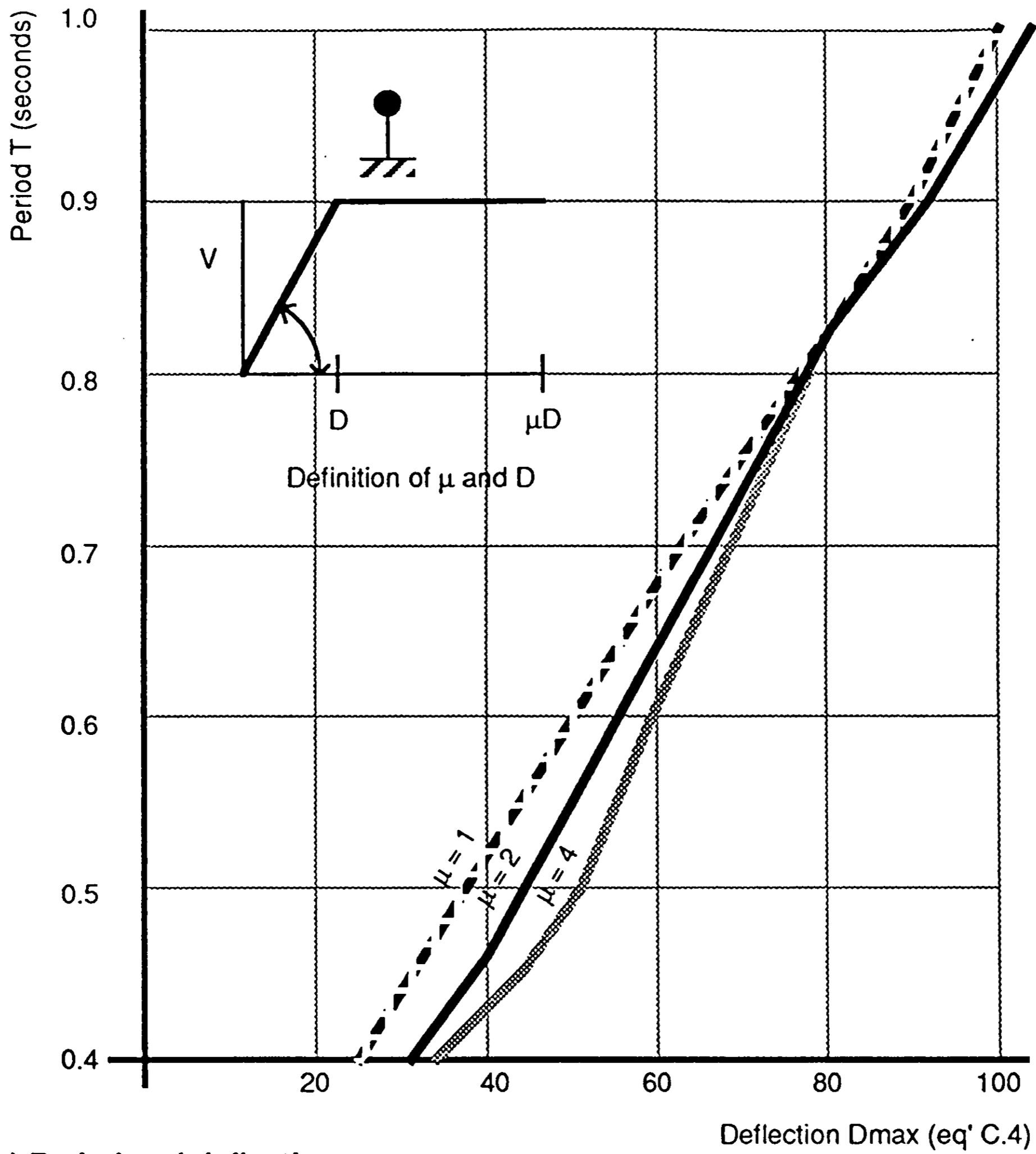


Ultimate lateral load capacity of short piles in cohesive soils.

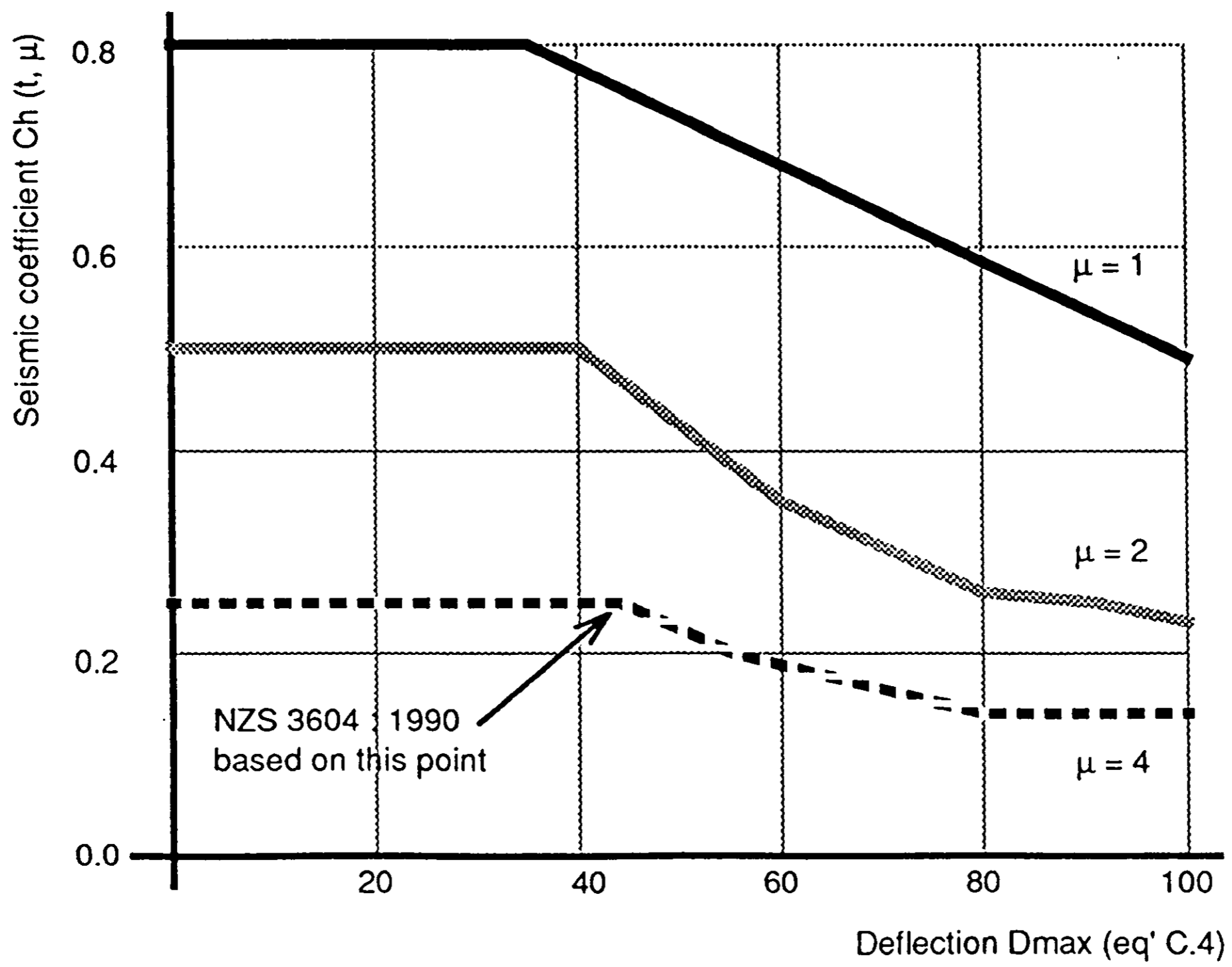


Ultimate lateral load capacity of short piles in cohesionless soils.

Figure B.4: Short pile solution after Works (1990)

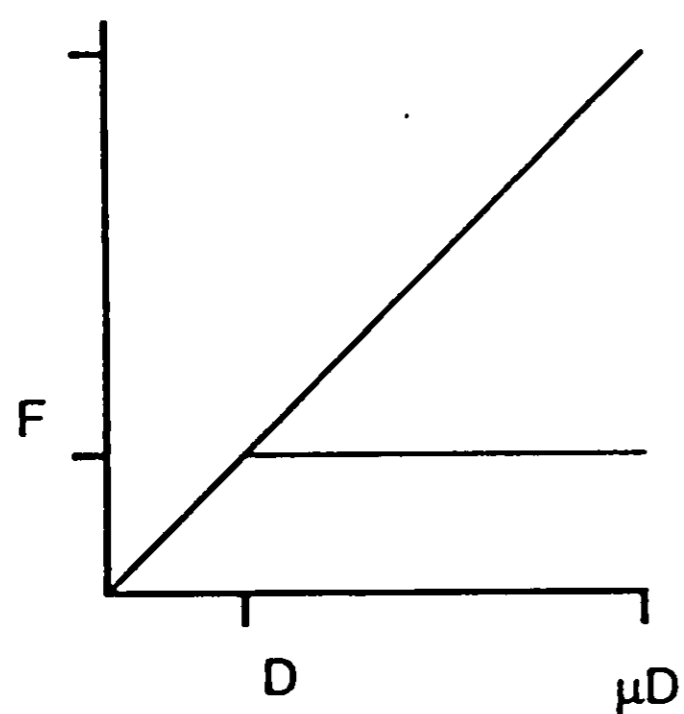


(a) Period and deflection

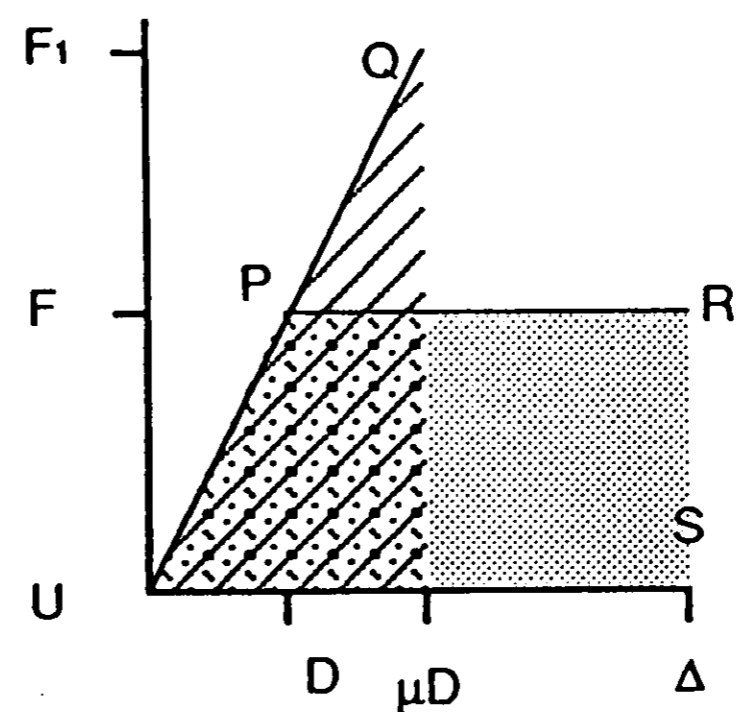


(b) Seismic Coefficient and deflection

Figure C.1 : Relationship between deflection, ductility period and seismic coefficient for a SDOF Wellington building



(a) Equal displacement principle



(a) Equal energy principle

Solution for equal energy principle :

From similar triangles : $F_1 = \mu F$

Area under elastic curve

$$= QTU = \mu F \cdot \frac{\mu D}{2} = F \mu^2 \frac{D}{2}$$

Area under bi-linear curve :

$$= UPRS = \frac{FD}{2} + F(\Delta - D)$$

Equating areas :

$$\frac{FD}{2} + F(\Delta - D) = F \mu^2 \frac{D}{2}$$

$$\text{or } \Delta - D = \frac{(\mu^2 - 1)D}{2}$$

$$\text{or } \Delta = \frac{(\mu^2 + 1)D}{2}$$

The ratio of maximum deflections from the two methods.

$$\begin{aligned} \frac{\Delta}{\mu D} &= \frac{(\mu^2 + 1)}{2\mu} \\ &= 1 \text{ for } \mu = 1 \\ &= 1.25 \text{ for } \mu = 2 \\ &= 2.125 \text{ for } \mu = 4 \end{aligned}$$

Figure C.2 : Equal area and equal displacement rules

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Design strength of various
house pile foundations



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HEAD OFFICE AND RESEARCH CENTRE

Moonshine Road, Judgeford
Postal Address - Private Bag 50908, Porirua
Telephone - (04) 235-7600, FAX - (04) 235-6070

REGIONAL ADVISORY OFFICES

AUCKLAND

Telephone - (09) 524-7018
FAX - (09) 524-7069
118 Carlton Gore Road, Newmarket
PO Box 99-186, Newmarket

WELLINGTON

Telephone - (04) 235-7600
FAX - (04) 235-6070
Moonshine Road, Judgeford

CHRISTCHURCH

Telephone - (03) 366-3435
FAX - (03) 366-8552
GRE Building
79-83 Hereford Street
PO Box 496