STRUCTURAL SYSTEMS
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STRUCTURAL RESPONSE ASSESSMENT OF SOIL NAIL WALL FACINGS

Executive Summary of Experimental and Analytical Investigations

by

F. Seible

Final Report to CH2M Hill Denver, CO, and the Federal Highway Administration under Demonstration Project DP-103.

February 1996

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DISCLAIMER

Any opinions, findings, conclusions, or recommendations in this report are those of the author and do not necessarily reflect the views of CH2M Hill or the Federal Highway Administration.

ABSTRACT

In support of the development of rational design models and guidelines for soil nail wall facings, the overall response behavior of soil nail wall facing panels under increasing nail load and/or nail head displacements was investigated to determine the behavior limit states that form the basis for LRFD and service design criteria. The experimental investigations conducted at the Charles Lee Powell Structural Research Laboratories at the University of California, San Diego, consisted of six full-scale single nail facing panel/soil structure interaction tests and one large scale nine nail facing panel/soil structure interaction test. The experimental test results were used to calibrate and validate a nonlinear analysis model which allowed the investigation of nonlinear facing panel material response with full soil structure interaction.

The validated analytical model was subsequently used for extensive parameter studies on temporary and permanent facing panels, investigating the influence of variations in soil reaction properties, facing panel material characteristics, facing panel dimensions, and nail head spacing and anchor plate dimensions.

The load/deformation response for temporary soil nail wall facing panels was found to be ductile in nature even when punching shear failure modes occurred due to the soil reaction under the nail head and membrane action provided by the facing panel reinforcement in the nail head vicinity. Only the overlaid permanent facing panel did not show signs of imminent failure due to internal delamination and punching failure not visible on the facing panel surface.

The analytical parameter studies showed that the nail load or facing capacity was most influenced by the facing panel thickness, the bearing plate size and the soil stiffness assumptions whereas reinforcement ratios and soil nail spacings primarily affected the deformation states which can be directly attributed to the highly coupled soil structure interaction effects.

1. INTRODUCTION

The Federal Highway Administration demonstration project DP-103 [1] had the objective to develop consistent and rational design procedures for temporary and permanent soil nail walls and soil nail wall facings, see Figs. 1 and 2. Temporary soil nail wall facings typically consist of thin shotcrete panels reinforced at the nominal panel mid-depth with welded wire fabric and waler bars, see Fig. 2, whereas permanent soil nail wall facing panels are typically cast-in-place in front of a temporary wall, see Fig. 1, with uniformly spaced mild reinforcement and headed stud anchors at the nail head plates, see Figs. 1 and 2. However, thicker permanent shotcrete facing panels for soil nail walls have been contemplated and used in some applications.

In any case, comprehensive design models for soil nail walls based on limit state design principles (LRFD or Service Limit States) require that the behavior limit states of all of the systems components, namely (1) the soil, (2) the nail, (3) the nail head, and (4) the facing panel are known to develop overall consistent and reliable design criteria and concepts. In order to assess the behavior limit states of the nail head and facing panel for a large range of realistic geometric and mechanical parameter variations, a nonlinear analysis model was developed [2, 3] which allowed, in particular, the study of various soil reaction assumptions and their complex interaction with the nonlinear material response of the soil nail wall facing panel in the form of cracking and crushing of the concrete, as well as yielding of the reinforcing steel.

Prior to broad based parameter studies of the soil structure interaction limit state behavior of soil nail wall facing panels, the developed analytical model was calibrated with full-scale tests on single nail/soil supported facing panels [4, 5] and verified by a full-scale nine nail facing panel test [6] in the Charles Lee Powell Structural Research Laboratories at the University of California, San Diego. The validated computer model was subsequently used to investigate the influence of different design parameters and variations on the soil nail wall facing panel response [7], and for specific case studies of commonly encountered temporary and permanent soil nail wall facing geometries and soil conditions [8].

It is the objective of this report to summarize both, the analytical model development and parameter studies, as well as the full-scale experimental investigations used for the validation of the analytical model in direct support of the soil nail wall design guideline development under FHWA DP-103. Detailed information on the analytical model development, the full-scale single and nine nail tests, and the parameter and case studies can be found in the referenced reports.

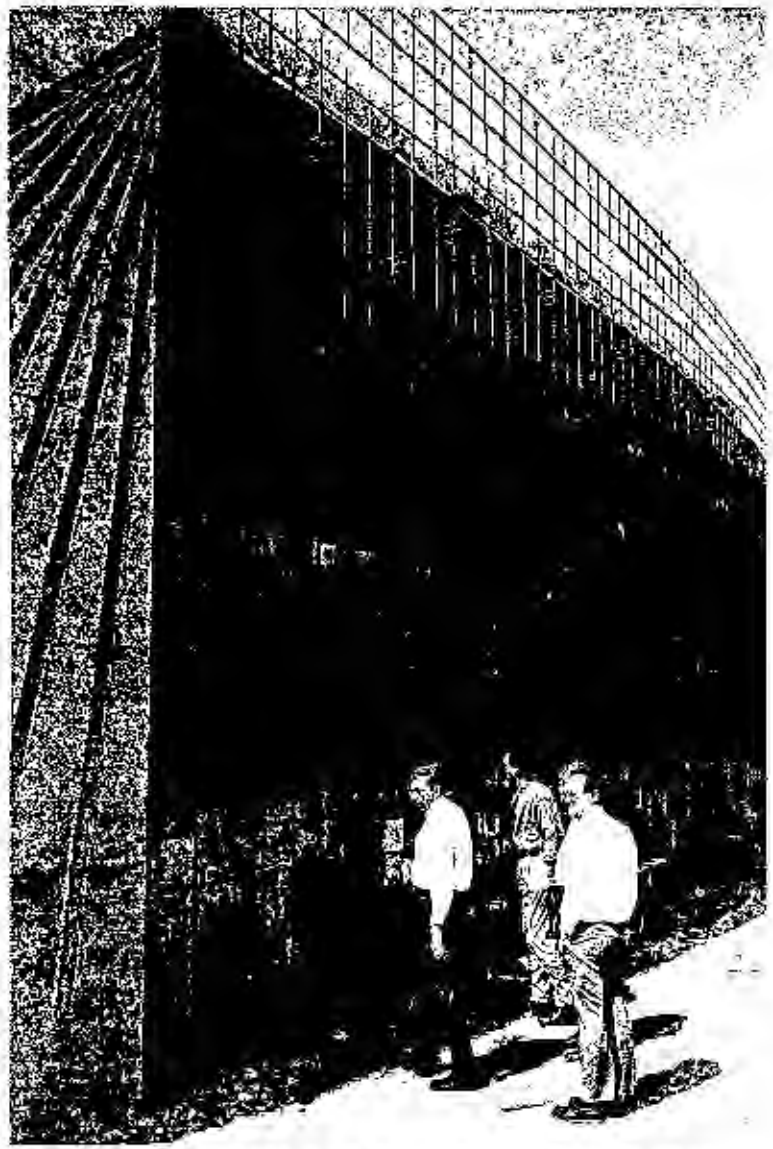


FIG 1. Permanent Soil Nail Wall Construction

FIG 2. Temporary Soil Nail Wall Prior to Permanent Facing Application

2. THE EXPERIMENTAL TEST PROGRAM

The experimental test program consisted of two types of tests, namely (1) single nail tests, and (2) a nine nail facing panel test. All tests were conducted at full-scale or close to typically encountered prototype scales. Since each test was a one-of-a-kind test no statistical value can be assigned to the test program, rather, the experimental test programs had the principal objective to allow calibration and verification of the developed analytical model. Such a model validation consists of two phases, namely (1) the calibration phase where the model is used to trace experimental response results to determine if the model has sufficient parameters to allow the phenomenological description of the observed behavior with parameter adjustments based on measured material properties and rational material models, and (2) the verification phase where the model is used to predict experimental behavior prior to the actual test.

For the calibration phase of the soil nail wall facing model a series of six single nail facing panel tests was conducted in a specially designed and constructed soil box test apparatus, see Fig. 3, where single soil nails are pulled through solid supported facing panels. The final model verification was performed via a nine nail test, see Fig. 4, where the nine nail arrangement simulated the facing panel continuity and thus more realistic boundary conditions than those modeled in the single nail tests with a free panel edge. Both test programs are summarized in the following.

2.1 Single Nail Facing Panel Tests

The single nail facing panel test program consisted of one calibration panel (cast-in-place concrete) and five soil nail wall facing panel tests. The single nail calibration test [4] was designed and performed as shake-down test for the soil box test apparatus and the test instrumentation as well as a first benchmark for the analytical model development. The five production single nail tests represented commonly encountered facing panel design/geometry/support differences to provide a representative behavior response variation for the analytical model calibration. Variations in the five production tests ranged from temporary shotcrete facings to a permanent (overlaid) facing panels, different dimensions of the anchor plate and different reinforcement types (i.e. with and without waler bars), as well as differences in the support conditions to investigate different failure modes. An overview of all six single nail facing panel tests is provided in Table 1.

The single nail facing panel test was designed to simulate realistic soil structure interaction under the nail head but with free boundary condition along the panel edges to simplify testing and test data interpretation. This simplification is justified by the single nail test objective of analytical

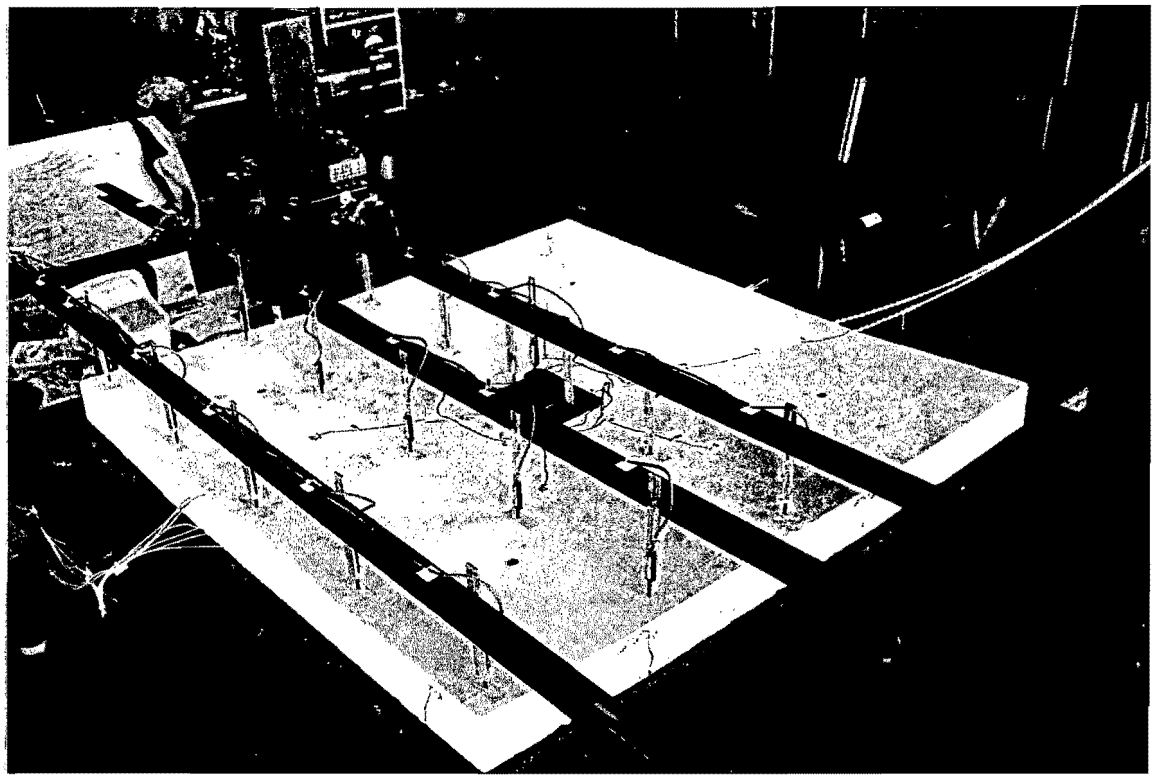


FIG 3. Single Nail Facing Panel Test

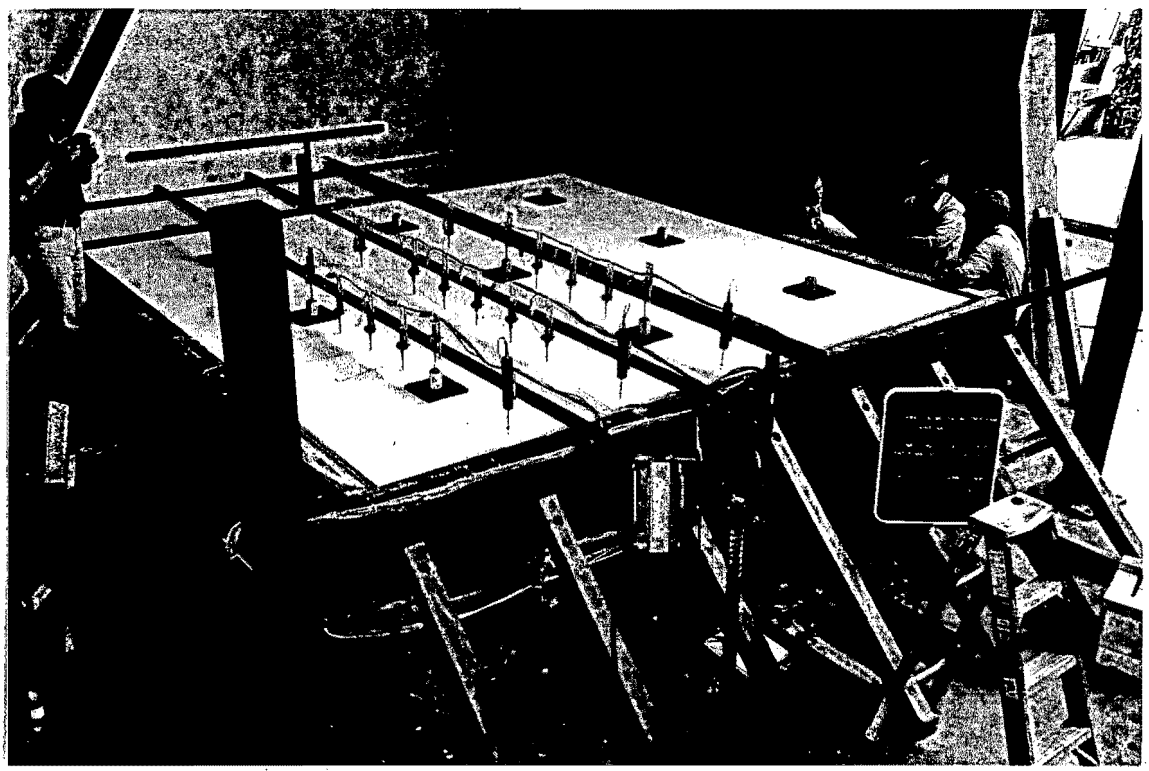


FIG 4. Nine Nail Facing Panel Test

TABLE 1. Single Nail Facing Panel Test Matrix (1.5 x 1.5 m panels)

Test Specimen	Panel Type	Support Condition	Panel Reinforcement		Anchor Plate [A36] Dimensions [mm]	Concrete Strength f'c [MPa]
Shake-Down Test	100 mm precast	compacted soil	Grade 75 welded wire mesh 6 x 6 W2.0 x 2.0	Grade 60 waler bars	13 x 200 x 200	27.6
Production Test No. 1	100 mm temporary shotcrete	2 x compacted soil	6 x 6 W2.1 x 2.1	—	13 x 200 x 200	24.1
Production Test No. 2	100 mm temporary shotcrete	4 x compacted soil	6 x 6 W2.1 x 2.1	—	25 x 200 x 200	24.1
Production Test No. 3	100 mm temporary shotcrete	3 x compacted soil	6 x 6 W2.1 x 2.1	2 #4 e.w.	13 x 200 x 200	24.1
Production Test No. 4	100 mm temporary shotcrete	shear ring 0.6 m diam.	6 x 6 W2.1 x 2.1	2 #4 e.w.	25 x 200 x 200	24.1
Production Test No. 5	100 mm shotcrete +200 mm cast-in-place overlay	5 x compacted soil	6 x 6 W2.9 x 2.9 + #6 @ 300 mm in overlay	2 #4 e.w.	19 x 200 x 200 with 4 headed studs	24.1 27.6 overlay

model calibration with subsequent more realistic (continuous) boundary conditions for the parameter studies and the nine nail verification test.

The geometry and dimensions of the single nail soil box are depicted in Fig. 5 and details on the design of the single nail facing panel test apparatus can be found in [4]. The test sequence consisted of (1) the construction of the shotcrete facing panels against plywood forms in the vertical position, (2) the placement of the panels on the compacted soil in the soil box, (3) the placement of the nail and nail head in the form of a high strength bar in a PVC sleeve and anchor plate, (4) the instrumentation with displacement transducers and soil pressure cells at strategic panel locations, (5) the testing of the panel by pulling the nail/nail head against the facing panel, (6) the observation of the failure mode subsequent to the test, and (7) the test data reduction and reporting.

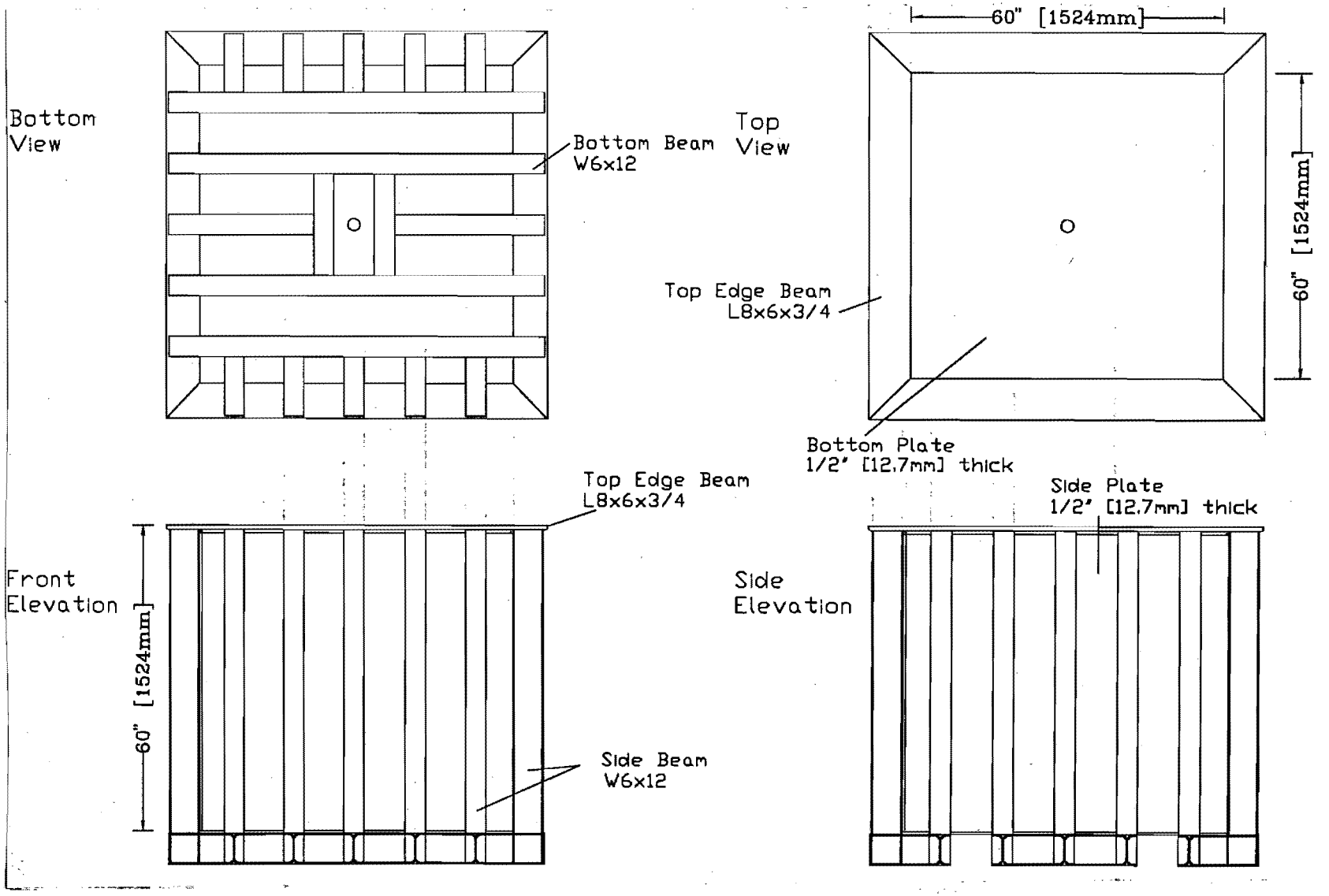


FIG 5. Geometry and Dimensions — Soil Nail Test Box

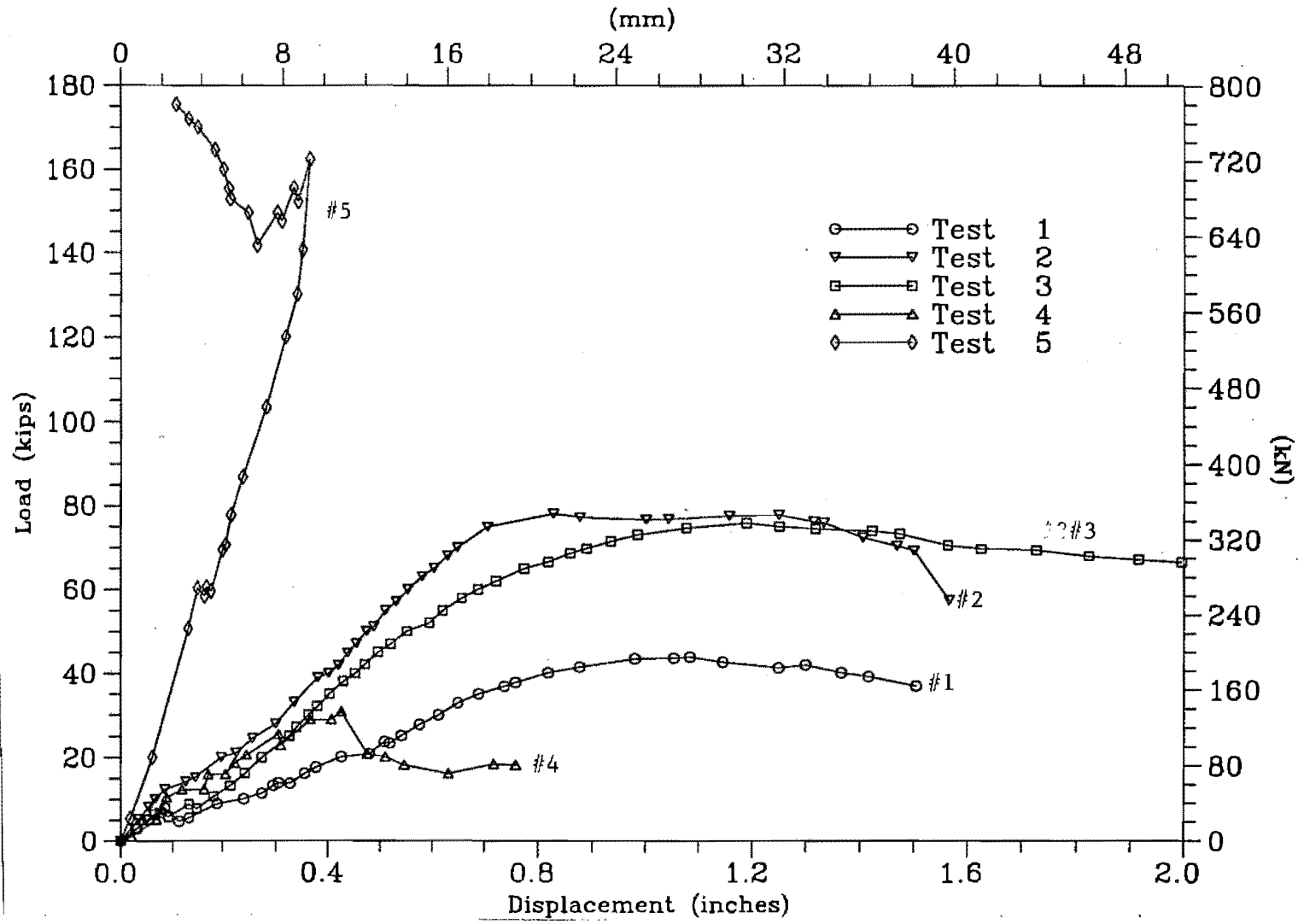


FIG 6. Nail Head Load vs. Displacement Response Envelope

The single nail production tests [5] showed the following key response characteristics:

1. The nail head load-deformation response of the temporary shotcrete facing panels (tests #1, 2 and 3) was very ductile as can be seen from Fig. 6. All three panels failed in a combined flexural and punching shear failure.

2. Panel #4 which was not supported by soil but by a 0.6 m Ø ring of elastomeric pads failed as expected in punching shear at a nominal punching shear stress level of $0.25\sqrt{f'_c}$ [MPa] with reference to the total facing panel thickness as will be discussed later. However, even in test #4 the nail load capacity stabilized with increasing panel deformations subsequent to an initial capacity loss at the onset of the punching failure, see Fig. 6, due to membrane action in the reinforcement under the nail head.

3. The permanent (300 mm thick) facing panel showed the expected higher stiffness and nail load capacity in Fig. 6, but since the deformations were measured against the permanent facing panel surface, delamination between the temporary shotcrete facing and the permanent overlay actually showed a reduction in panel deformation after the internal punching shear failure which resulted in a potential failure mode without visual warning in the form of large facing panel deformations, a failure behavior which needs to be appropriately reflected in the design guidelines.

4. Facing panel nail line deformations depicted in Fig. 7 clearly show the influence of added waler bars, anchor plate thickness, panel thickness and soil reaction stiffness. Since each subsequent test compacted the soil in the single nail box more, a direct comparison of test results needs to consider this difference in soil reaction stiffness. As can be seen from Fig. 7, significant uplift along the panel edges occurred in tests #1 and 3 where the soil compaction was at a minimum. Note that the tests #2 and 3 were not performed in sequence, but rather test #3 before test #2!

5. The soil reaction stiffness measurements in Fig. 8 also show the continued compaction effect from each test. Furthermore, since each single nail panel was placed in cured form in the soil box and not shot against the soil, an initial settlement or soft branch is noticeable in all soil stiffness measurement. At higher deformation levels for the panels where uplift of the corners and panel edges occured, Fig. 8 also shows a softening branch at the end of the test which is indicative of soil failure under the nail head. This soil failure occurs only in cases where the panel uplift no longer provides soil confinement. Trilinear soil stiffness characteristics derived from the single nail tests as part of the analytical model calibration are also depicted in Fig. 8.

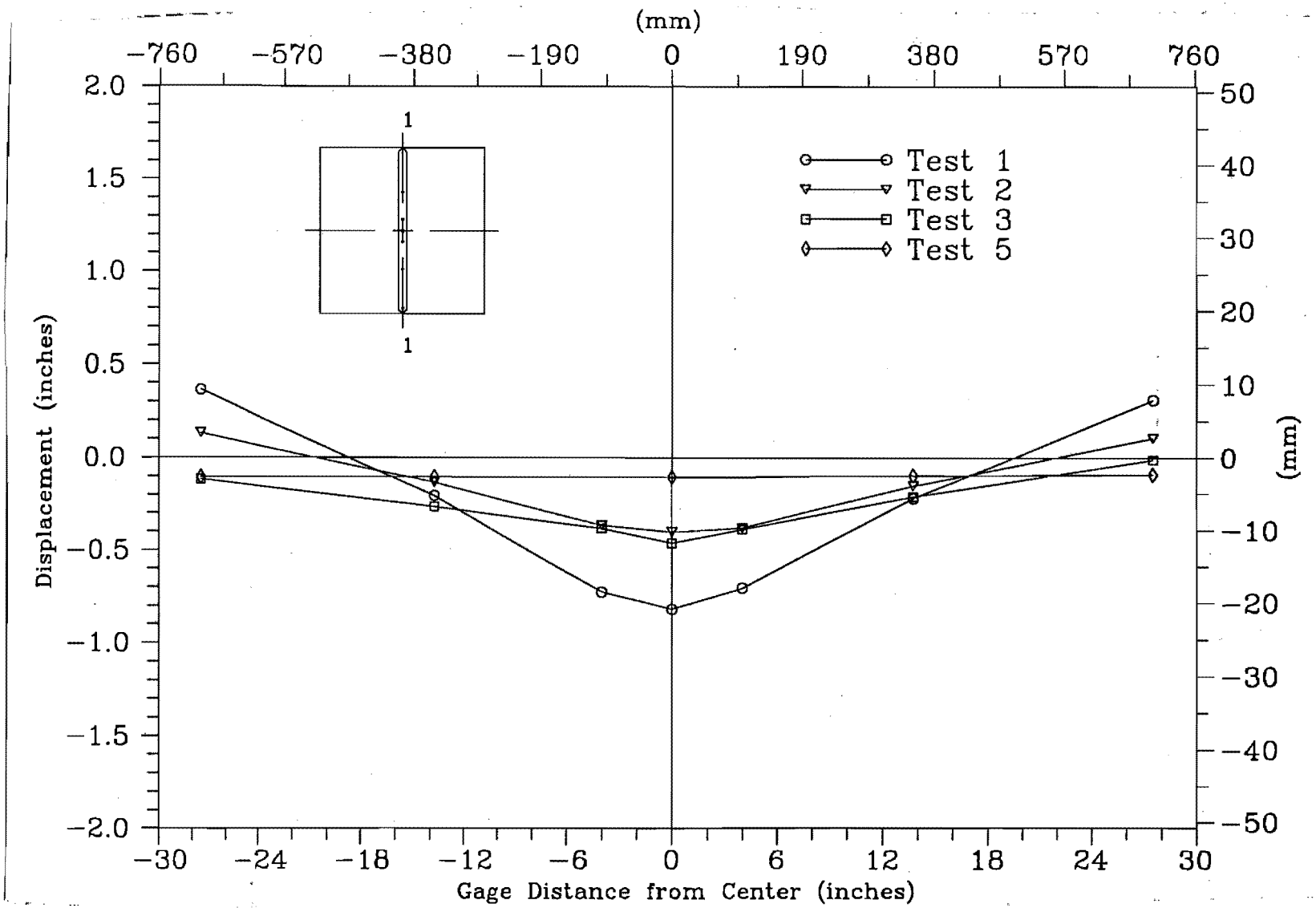


FIG 7. Surface Displacement Profile 1 at 40 kips Concentrated Load

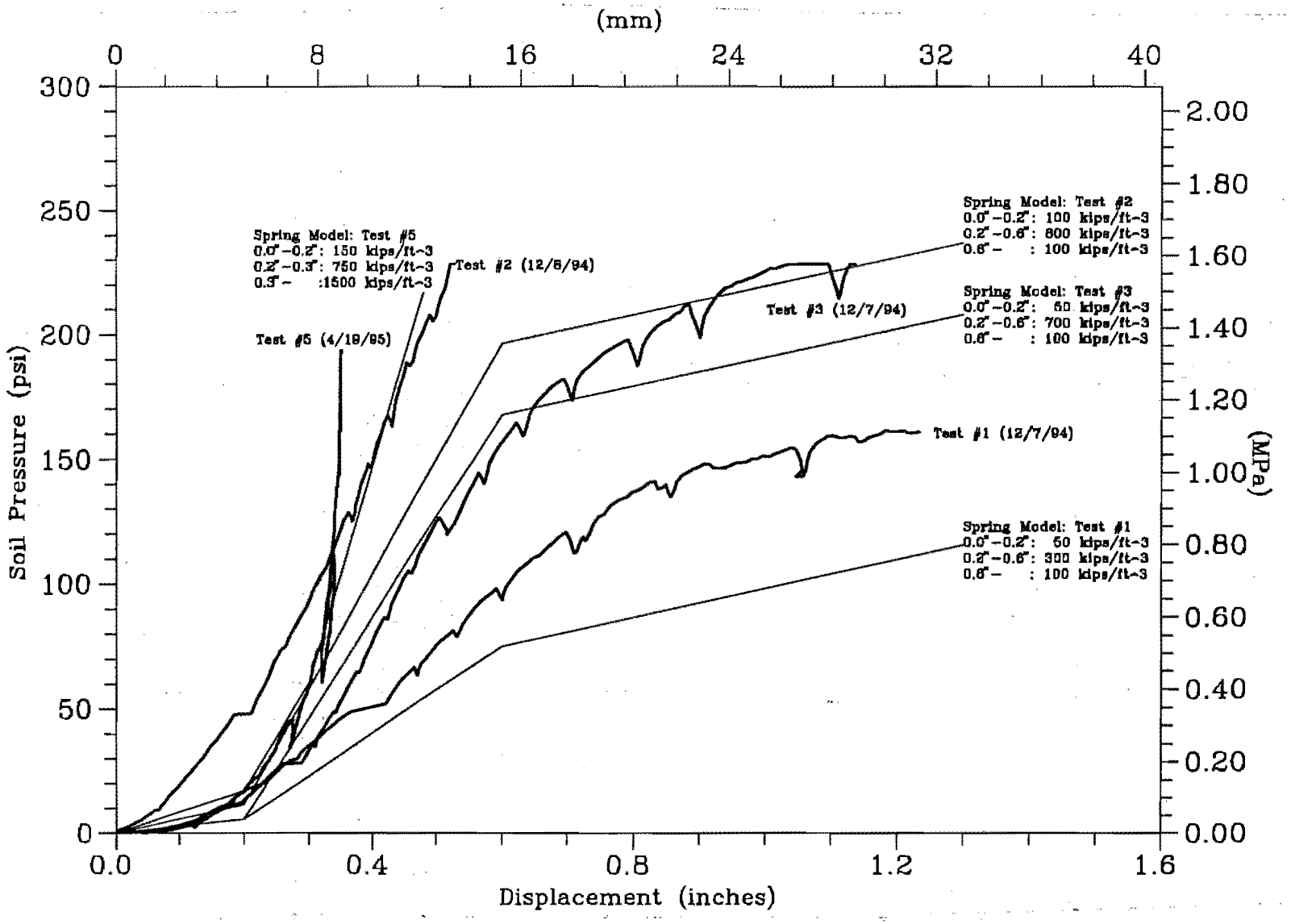


FIG 8. Soil Pressure vs. Center Displacement

6. The failure modes in the temporary shotcrete facing panels loaded against soil can be characterized as flexural-tension-shear failures which can be explained as follows. First, radial flexural cracks originate from the nail head at the soil side of the facing panels. Second, a concentric radial tension field develops around the nail head indicated by a circumferential crack pattern, see Fig. 9, with radius around the nail head influenced by the relative soil and facing stiffness. Finally, punching shear failure originating from the anchor plate and spreading with an approximate 45° failure cone through the panel depth forms the third failure mechanism, see Fig. 10. This sequence of three mechanisms as well as the membrane action provided by the reinforcement and the support provided by the soil under the nail head all result in very large deformations and noticeable distress prior to any significant capacity degradation which makes the temporary shotcrete facing failure a very ductile failure mode. On the other hand, the above discussed punching shear failure and delamination in the overlaid permanent facing panel occurred without prior warning but at very high nail loads.

7. Punching shear failures, though not as brittle as discussed above, were found to occur at nominal shear stress levels of $0.25\sqrt{f'_c}$ [MPa] when referenced to a nominal vertical failure plane through the full panel depth, where the 45° punching cone originating from the anchor plate intersects the center of the facing.

2.2 Nine Nail Facing Panel Test

The nine nail facing panel test [6] was conducted as final validation test for the analytical model. The plan geometry of the nine nail test is depicted in Fig. 11 with a 1.20 m nail spacing in both directions. The depth of soil was also 1.20 m based on St. Venant's assumption of a uniform stress state in a homogeneous medium.

The facing panel construction differed from the single nail facing panel test in that the 100 mm shotcrete panel was directly shot onto the sand filled box thus consolidating the sand during the casting operation and ensuring good contact between the soil and the facing panel. The nominal concrete strength for the panel was 27.6 MPa but at the day of testing the actual concrete strength was 50% higher. The reinforcement of the 100 mm thick temporary facing panel consisted of 6 x 6 W2.9 x 2.9 welded wire fabric and 2 #4 waler bars each way at each nail head line. The anchor plate dimensions were 19 x 203 x 203 mm.

All nine nails were uniformly loaded through a common manifold up to the onset of punching shear failure. Subsequently, the external nail loads were locked-off and kept constant while the center nail (nail #5, Fig. 11) was continued to be pulled through the facing panel.

FIG 9. Distress Pattern at Top of Test Panel #2

FIG 10. Punching Shear Distress Pattern at Bottom of Test #1 Panel after Test

General observations from the nine nail test can be summarized as follows:

1. The load-deformation behavior in general and for the center nail in particular, was very ductile as depicted in Fig. 12, consistent with the observations from the single nail tests. The analytical prediction of the load-deformation behavior of the center nail head was offset from the test results by the amount of the reduced initial soil stiffness as observed and implemented based on the single nail test results. However, the different construction of the nine nail panel by shotcreting against the soil eliminated this initial reduced soil stiffness phase. Furthermore, the free panel perimeter edge still allowed some uplift whereas in the center nail vicinity continuous confinement was provided to the soil by the facing panel. Changing the trilinear soil reaction stiffness characteristics used in the predictive model to linear characteristics but with reduced stiffness towards the outer regions of the nine nail facing panel, as shown in Fig. 12, resulted in a diagnostic post-test analysis which correlated well with the experimental results.

2. The justification for the linear soil stiffness characterization was also provided by the soil pressure measurements between nails 4 and 5 which are reduced to a pressure/deformation relationship in Fig. 13 for the center nail location. The actual soil pressure profile between nails 5 and 4 is depicted in Fig. 14 for various nail load levels. Note that after a nail load of 311 kN only the center nail #5 was loaded while all the other nail loads were kept constant.

3. The crack pattern evolution in the nine nail test on the exposed side of the facing panel is depicted in Fig. 15 for various nail load levels. The first visible cracks develop in the negative moment regions between nail lines, followed by circumferential tension cracks around the nail heads and ultimately punching shear cracks originating from the anchor plate perimeter. Thus, the sequential distress patterns are consistent with the findings from the single nail tests at slightly higher nail loads due to the panel continuity. The above described distress patterns are also depicted in Fig. 16 for the circumferential tension mechanisms and in Fig. 17 for the punching shear failures.

4. Nominal punching shear stress levels as described for the single nail facing tests slightly exceeded $0.4\sqrt{f'_c}$ [MPa] for the center nail and were thus 1.5 x higher than for the single nail tests, which can be attributed to the higher soil capacities in the continuous panel case.

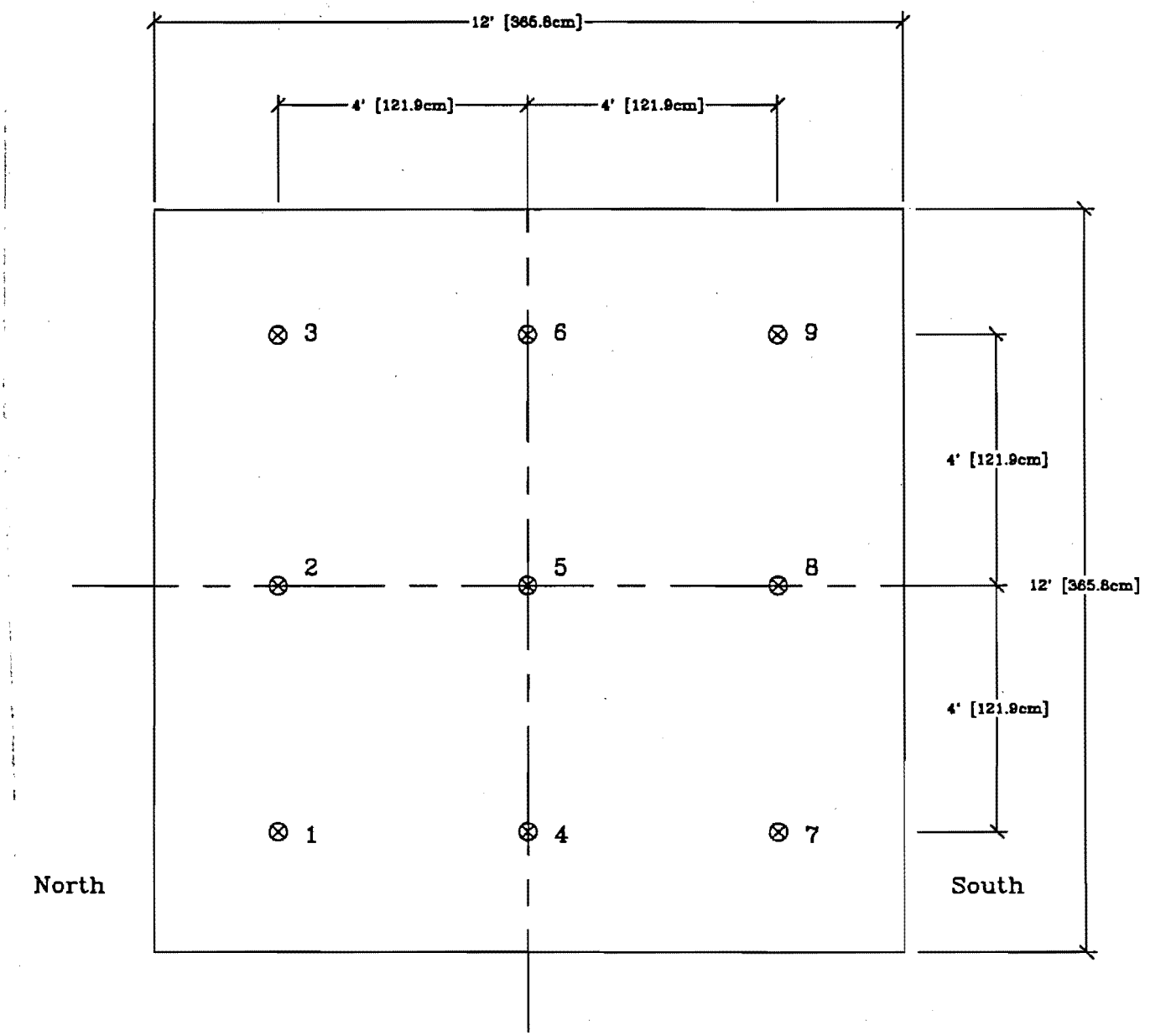


FIG 11. Nail Head Location and Numbering

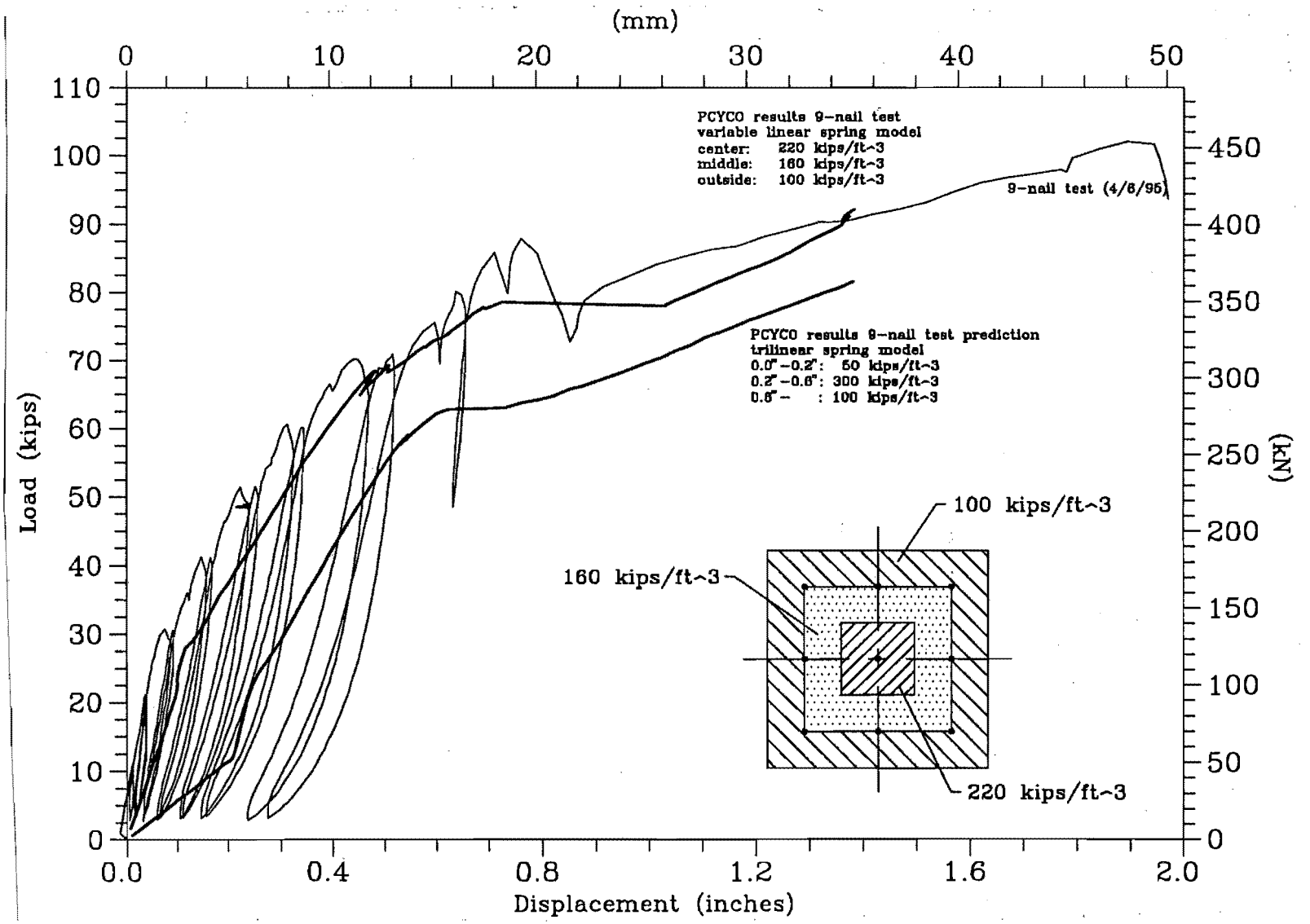


FIG 12. Nail Head Load vs. Displacement Response, 9-Nail Test

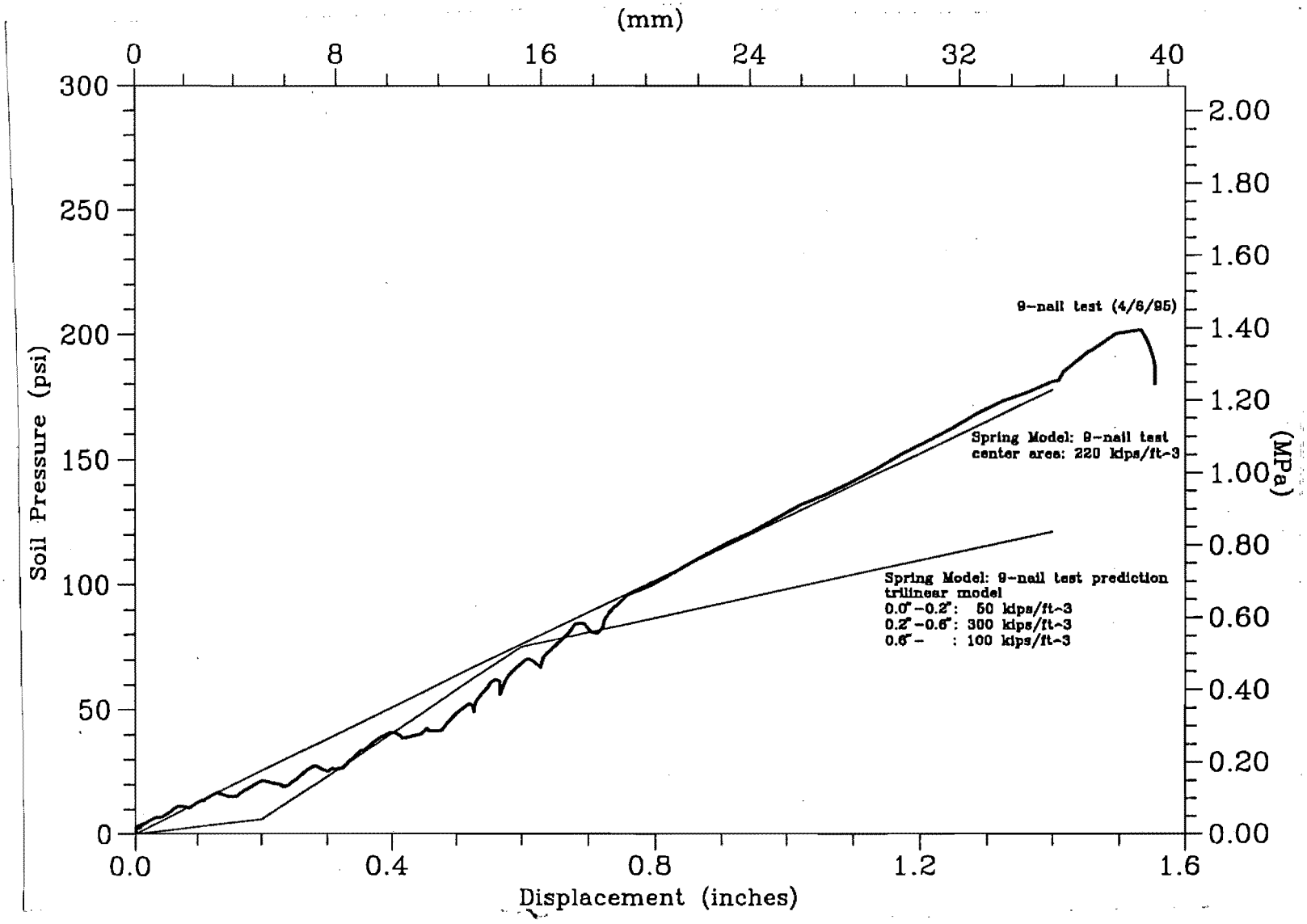


FIG 13. Soil Pressure vs. Center Displacement, 9-Nail Test

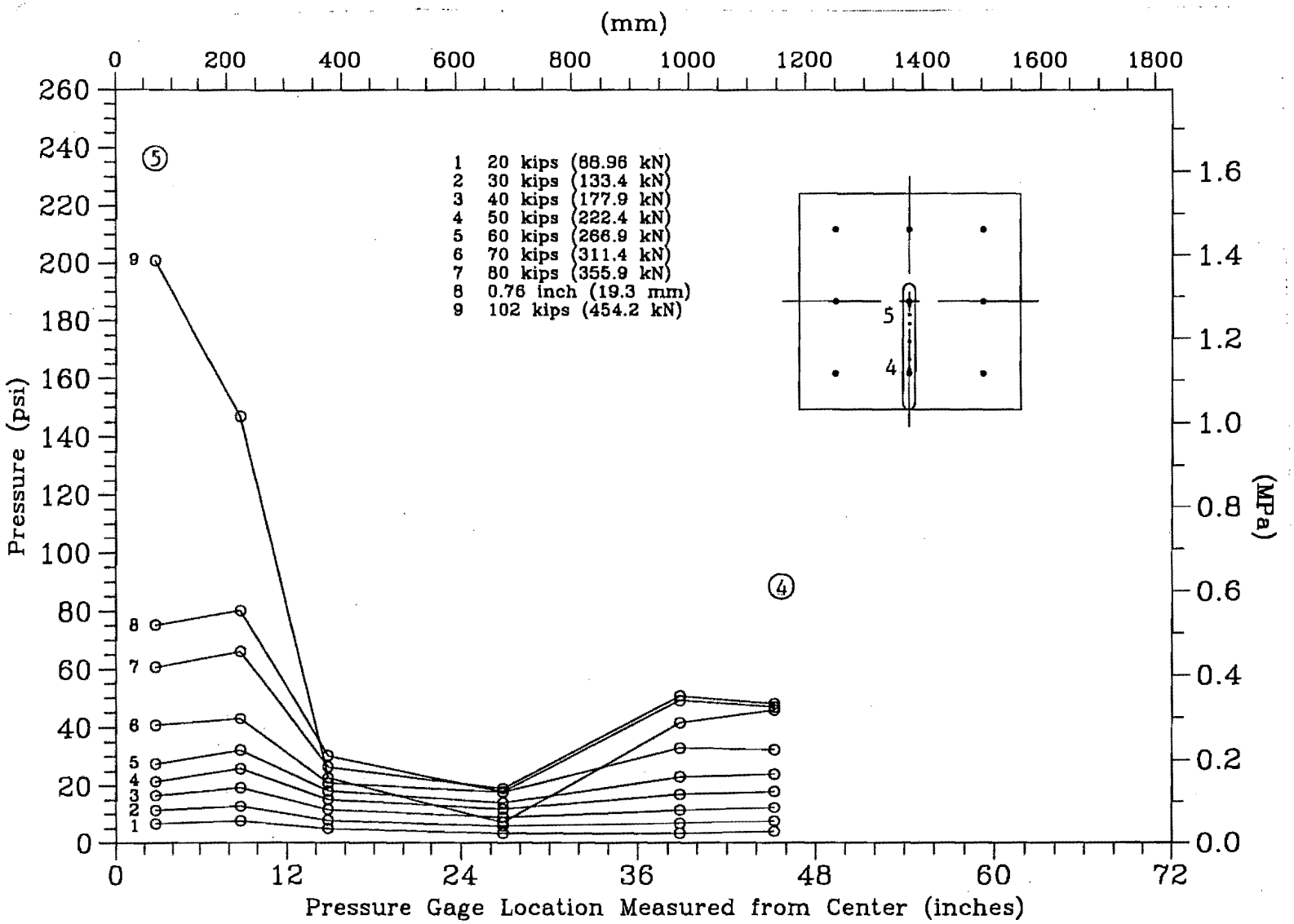
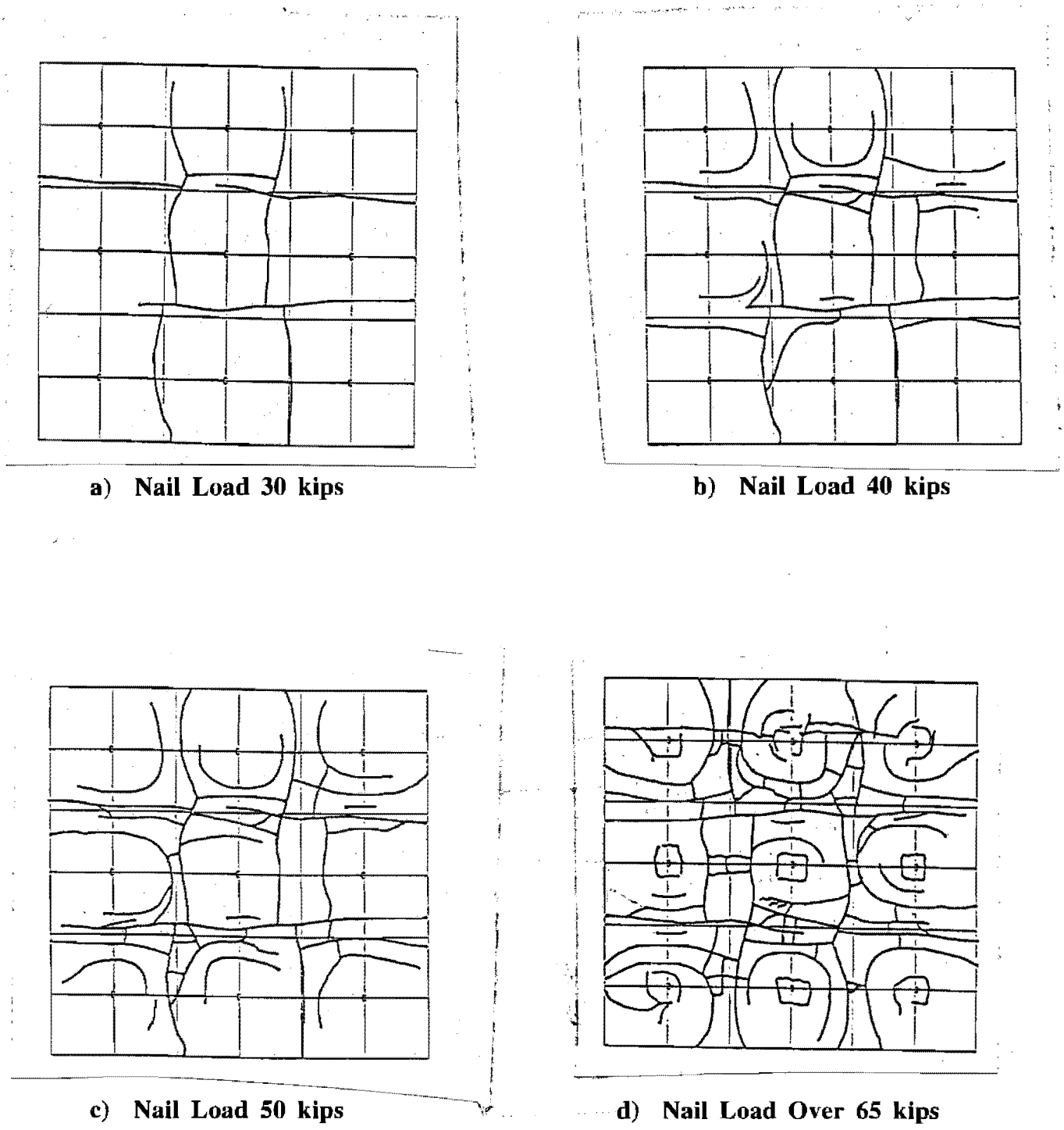


FIG 14. Soil Pressure Gage Profile



a) Nail Load 30 kips

b) Nail Load 40 kips

c) Nail Load 50 kips

d) Nail Load Over 65 kips

FIG 15. Top Surface Crack Pattern

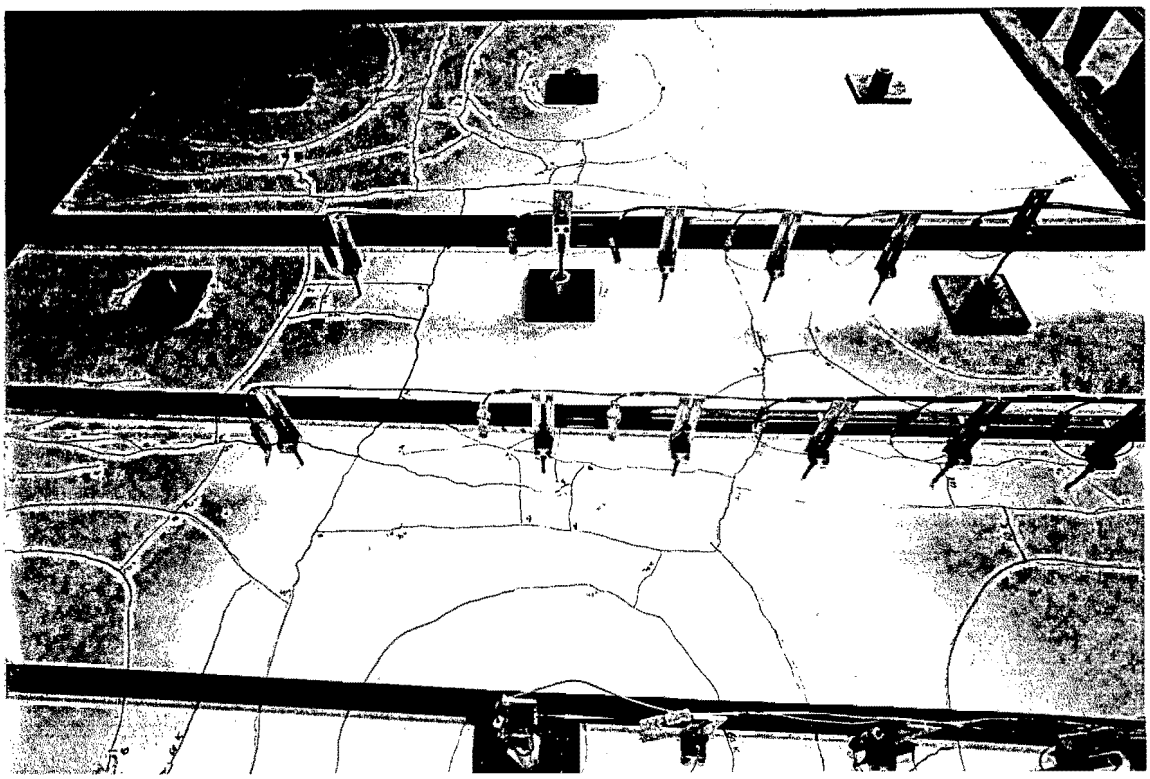
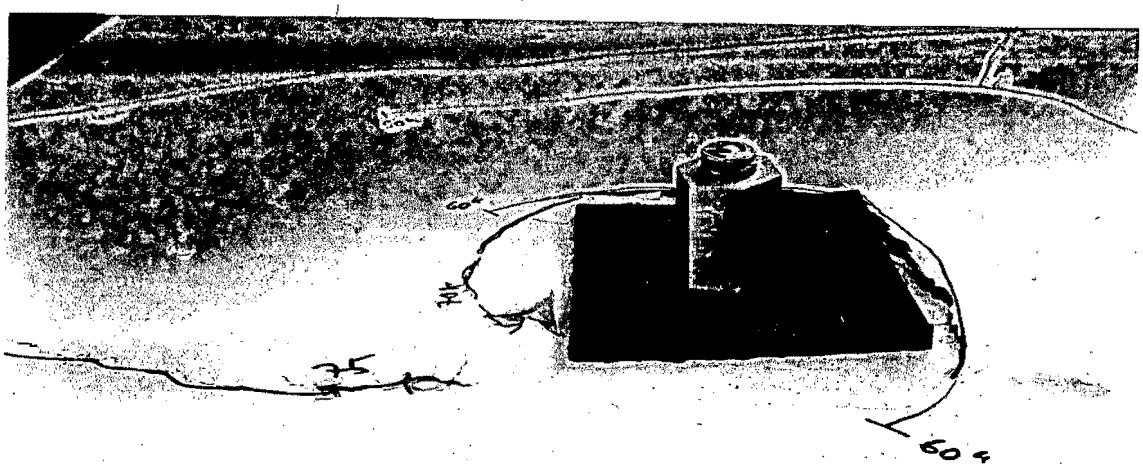
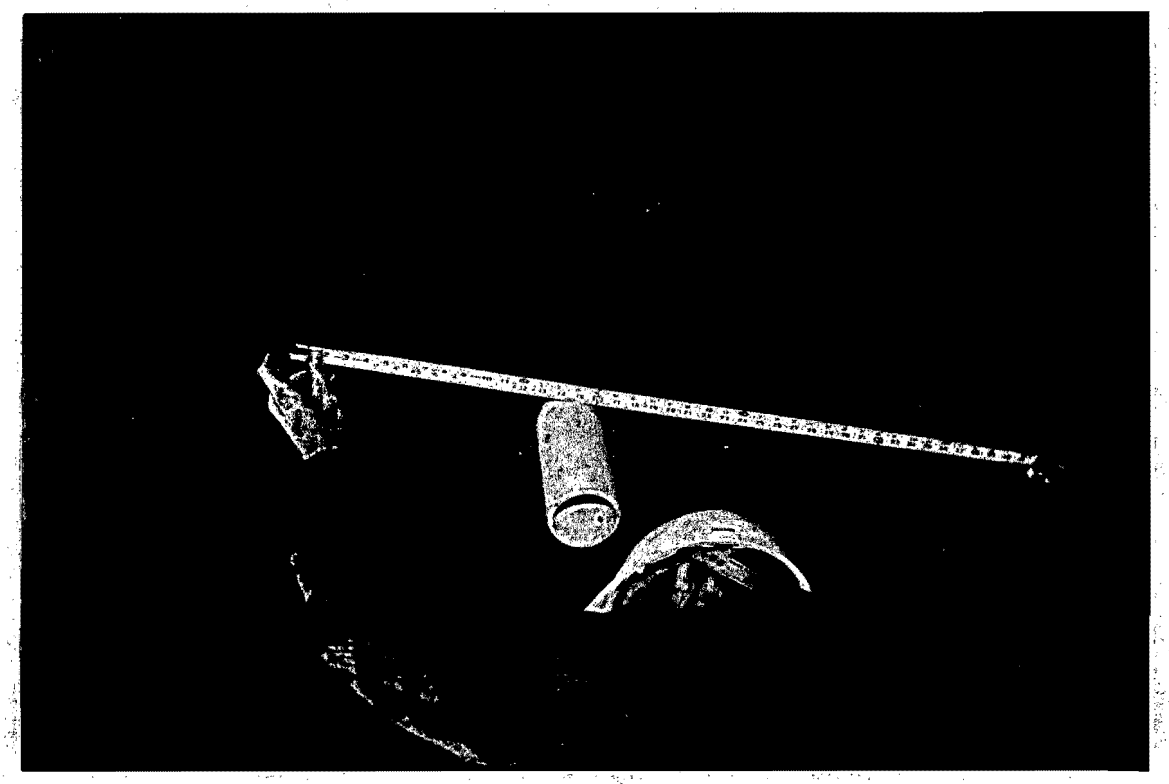


FIG 16. Top of Panel, Final Distress Pattern



a) Punching Failure Top and Bottom of Panel



b) Crack Pattern and Imprint of Center Nail Punching Cone

FIG 17. Close-up of Facing Panel Failure Modes

3. THE ANALYTICAL INVESTIGATIONS

The analytical model development, parameter studies, and case studies are described in detail in [2, 3, 7 and 8] and are summarized in reference to the above experimental program in the following.

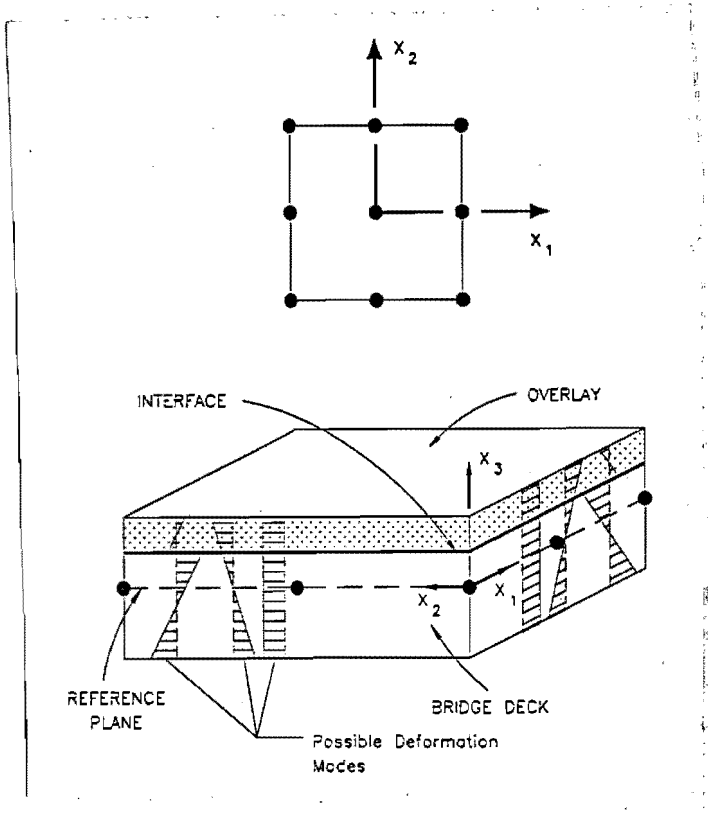
3.1 Analytical Model Development

For the detailed nonlinear analysis of soil nail wall facing panels, the computer code PCYCO [2] was adopted, since critical facing panel response limit states such as cracking of the concrete, yielding of the reinforcement, crushing of the concrete, etc., can be modeled.

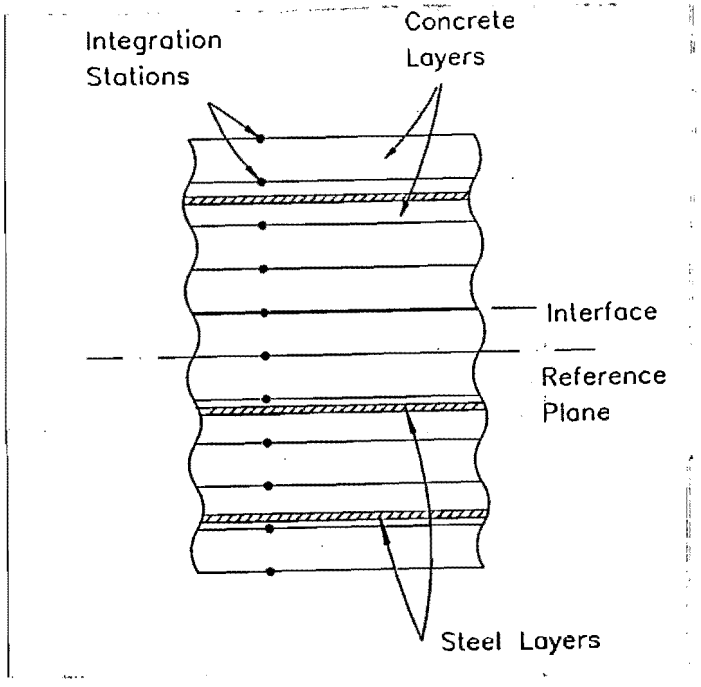
PCYCO is the latest in a series of nonlinear analysis programs for structural concrete members and systems developed at the University of California, San Diego, specifically for the prediction and diagnostic of reinforced or prestressed concrete or masonry systems test performed at the Powell Structural Research Laboratories. The PCYCO program is a research tool and not a production type program, hence code clarity was chosen over program efficiency whenever it was necessary.

The plate or slab element in PCYCO, which was used to model soil nail wall facings, is a nine-node Lagrangean isoparametric flat shell element in which the concrete is represented as a layered media and the steel reinforcement is overlaid and connected at the nodes by compatibility requirements, see Fig. 18. The basic plate bending behavior is modeled after the Reissner-Mindlin theory for shear deformable plates, and the constitutive model for concrete is based on the orthotropic model of Darwin and Pecknold, as well as on the modified compression field theory by Collins and Vecchio. An object-oriented programming philosophy has been adopted in the development of the PCYCO program. This programming philosophy considers the element kinematics independently of the material constitutive models. As a result, a researcher can develop a new constitutive model which can be inherited by the existing element types. Similarly, a researcher can create a new element type which can inherit any of the existing constitutive models with minimal programming effort. A common output data base ensures that element deformations, forces, strains, and stresses of a new element type can easily be incorporated into the existing post-processing programs.

The following material nonlinearities can be considered: (a) cracking of concrete, (b) softening and crushing of concrete, (c) yielding and strain hardening of reinforcement steel, (d) nonlinear stress-strain behavior including the Baushinger effect (cyclic model only), (e) material anisotropy,



a) Nine Node Element



b) Layering Through Element Depth

FIG 18. PCYCO Reinforced Concrete Flat Shell Element

(f) confinement effects in concrete, (g) tension stiffening effect of reinforced concrete (after cracking), and (h) prestressing effects on both steel and concrete. These effects are introduced in the analytical model through equivalent uniaxial orthotropic rotating constitutive relationships for each individual concrete or steel layer. The constitutive relationships for concrete and steel can either be linear, nonlinear elastic, or nonlinear inelastic and fully cyclic. The nonlinear elastic constitutive models were chosen for the subsequent soil nail wall facing analyses.

A preliminary model validation [3] 102 mm thick precast concrete single nail panel test [4] showed that the characterization of soil springs supporting the facing panel in the form of a linear elastic Winkler foundation needed some modifications to account for observed effects such as (1) uplift along the free edge of the single nail panel, (2) stiffening of the soil due to compaction during the test, and (3) softening of the spring support once the soil failure limit state is reached.

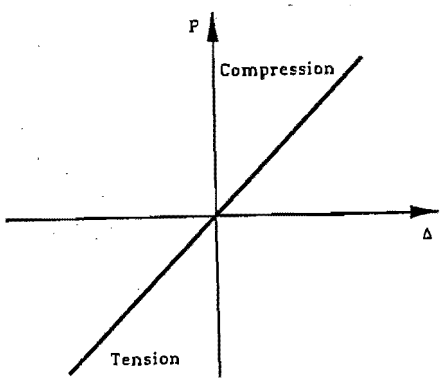
First, the linear elastic soil spring model, see Fig. 19a, was replaced with an elastic linear compression and tension gap model, see Fig. 19b, for uplift simulation. Then, a bilinear constitutive relationship for soil spring compression was introduced, see Fig. 19c, followed by refinement to a trilinear compression model to simulate the observed soil dislocation under the loading area at high nail loads. Finally, to model possible permanent soil pressure states against the back of the soil nail wall facing at higher wall depth, a cut-off pressure correction in the form of a baseline shift on the load in the soil spring model, see dashed horizontal axis in Fig. 19d, was introduced to the PCYCO program in direct support of the SSI (soil structure interaction) modeling of a concrete facing under increasing nail load/displacement limit states.

Based on load-deflection measurements in the single nail panel tests and parallel soil pressure cell readings during the test, both a calibration of the soil spring stiffness models described above, as well as a verification by direct comparison of the soil spring load-deformation behavior with the pressure cell readings were performed.

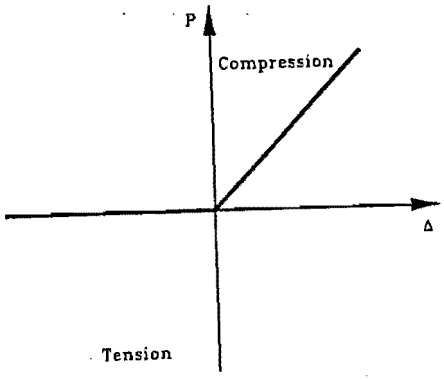
Analytical model discretizations used by both single and nine nail test analyses are depicted in Fig. 20, and show that due to symmetry of structure and loading only a quarter of each test panel needs to be modeled with appropriate boundary conditions along the lines of symmetry, allowing vertical displacements and rotations perpendicular to the boundary line and no rotations about the boundary axis.

a) The Single Nail Tests

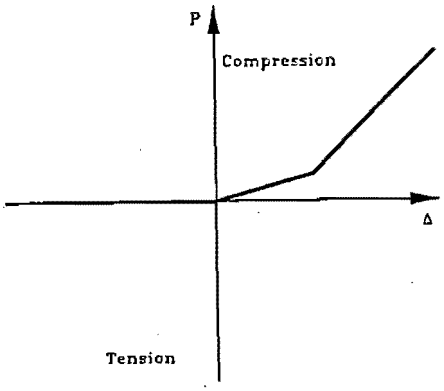
Subsequent to each single nail laboratory test, [5],the trilinear spring stiffnesses were adjusted such as to provide a close correlation with the observed load-deformation characteristics at the nail



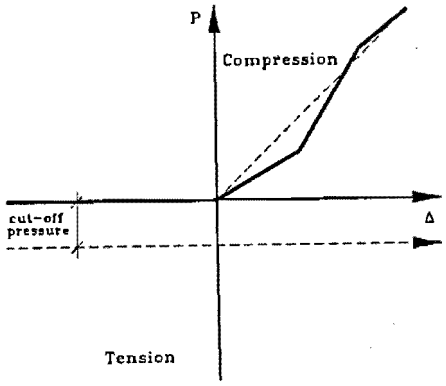
a) Linear Base Model



b) Linear Compression and Tension Gap Model

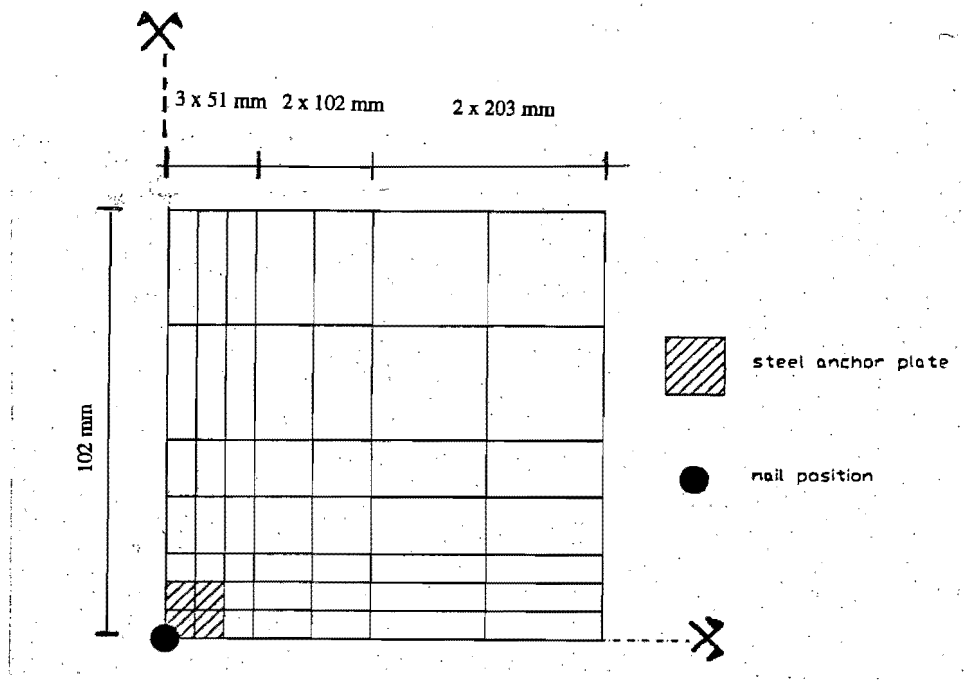


c) Bilinear Compression and Tension Gap Model

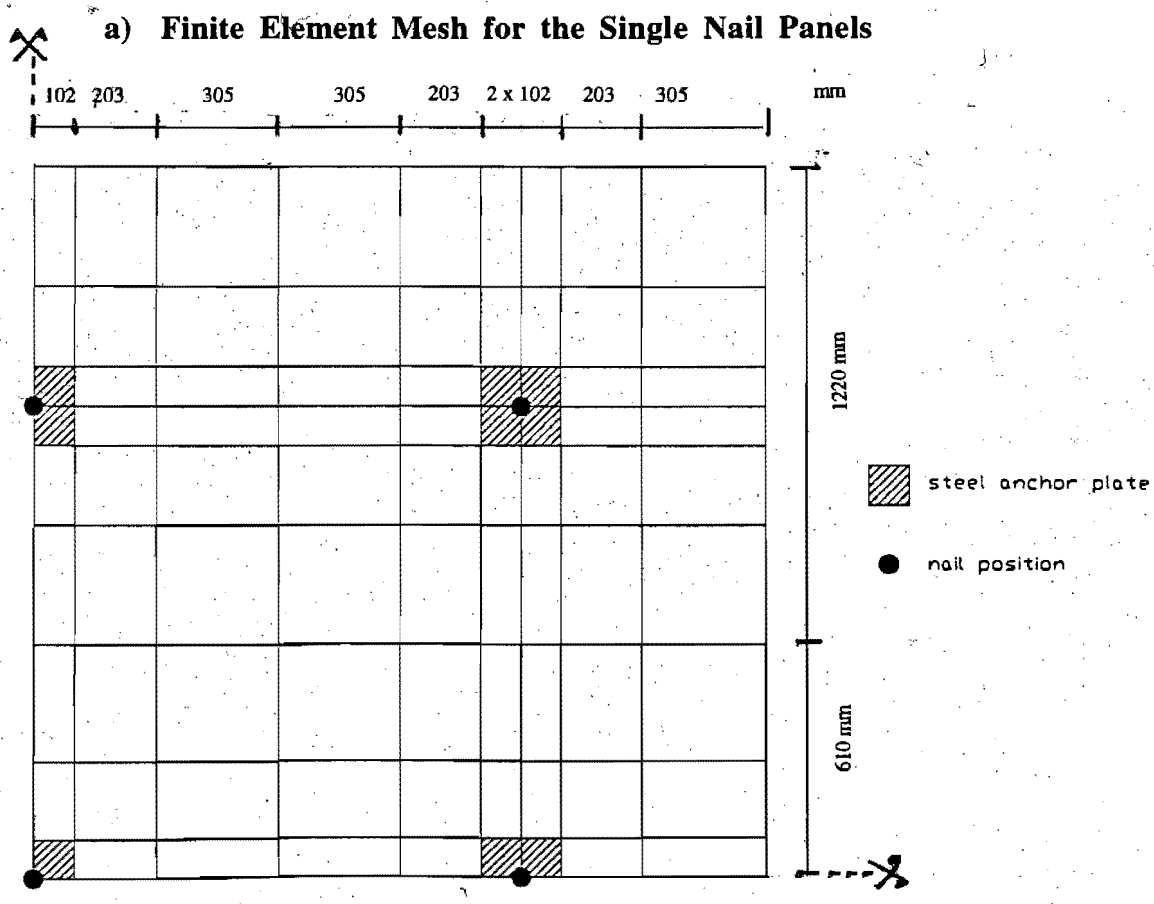


d) Trilinear Compression and Tension Gap Model with Same Line Shift

FIG 19. Elastic PCYCO 1-D Spring Model Development



a) **Finite Element Mesh for the Single Nail Panels**



b) **Finite Element Mesh for Nine Nail Panel**

FIG 20. Analytical Model Discretizations

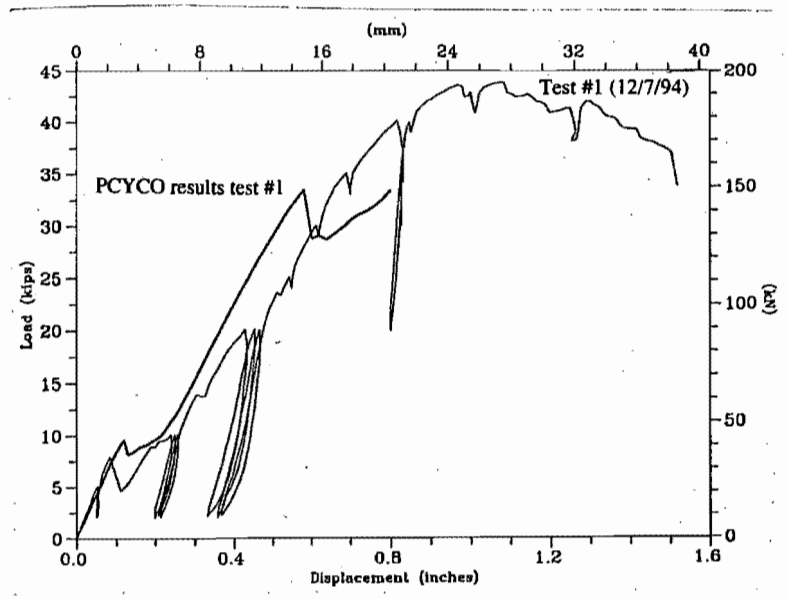
head. This calibration was performed for single nail test #1, 2, 3, and 5 only, since test #4 represented a punching shear test without soil support conditions or SSI.

The analytical post-test load-deflection traces for tests # 1, 2, 3, and 5, shown in Fig. 21, are in direct comparison to the experimentally observed nail head load-displacement curves. The fact that experimentally observed load-deflection curves can be traced after the test with the adjusted analytical model is in-itself no statement of qualification for the analytical model. It only shows that the model has sufficient parameters and characteristics which allow a good trace to be established. However, two additional conditions are very important to the model validation, namely (1) the fact that only soil spring stiffness were adjusted and measured material properties for f_c' = concrete strength, f_{cr} = the cracking strength as determined from standard cylinder and beam tests, and f_y = reinforcement yield based on bar pull tests or manufacturer supplied batch data were used and <u>not</u> varied as part of the model calibration, and that (2) measured soil pressures and deflection profiles along the soil pressure cell line allowed for an independent experimental verification of the analytically employed soil pressures or spring load vs. deflection data, as depicted already in Fig. 8. While the correlation in Fig. 8 between experimental and analytical data certainly could be improved, the general soil spring behavior characteristics has been verified. Based on Figs. 21 and 8, the validity of the trilinear model for the single nail facing panel tests was established. The trilinear break points in the soil spring stiffness are seemingly independent of soil condition and correspond to nail head displacements of 5.1 mm and 15.2 mm, respectively. The individual segment spring stiffnesses for the trilinear models listed in Fig. 8 correspond exactly to those used to generate the analytical traces in Fig. 21.

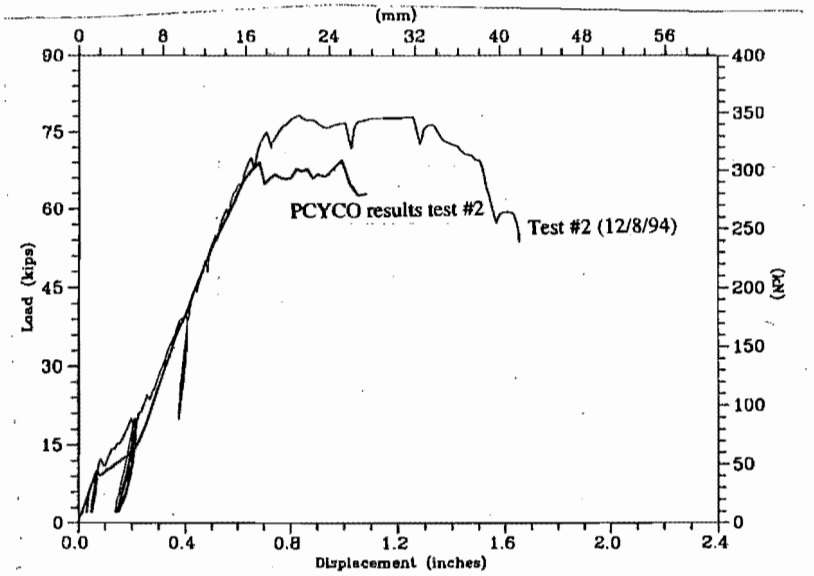
b) The Nine Nail Test

The calibrated analytical model was now used to predict the behavior of the nine nail test, already shown in Fig. 12, in a true class "A" or pretest prediction in order to validate the analytical tool prior to the detail parameter studies. A 7.8/47.0/15.7 $\frac{MN}{m^3}$ trilinear soil spring model was adopted for the nine nail test assuming that the density and compaction of the soil would resemble those of the first single nail test, see Fig. 8, test #1.

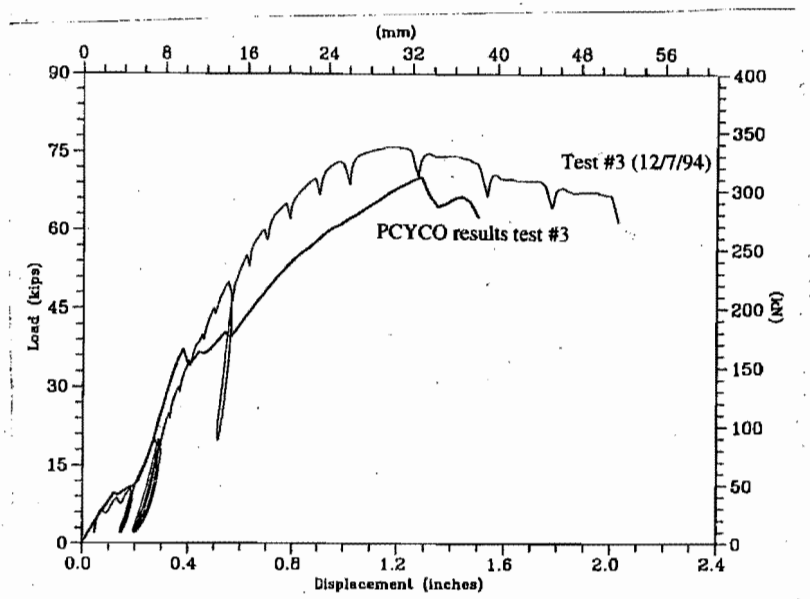
A direct comparison of the class "A" prediction and the experimentally measured load-deflection behavior at the center nail, depicted in Fig. 12, shows that the analytical model consistently over-predicts displacements for a given nail load level. On the other hand, once the initial soft spring stiffness up to a displacement of 5.1 mm is eliminated, very close agreement between experiment and analysis can be expected since it results effectively only in a lateral shift of the analysis curve towards the origin by an amount equal to the displacement during the initial



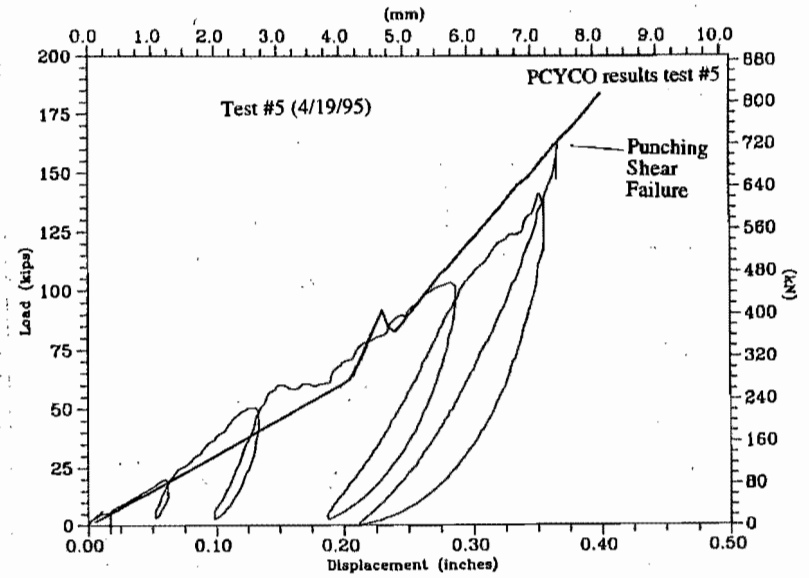
a) Test 1



b) Test 2



c) Test 3



d) Test 5

FIG 21. Nail Head Load vs. Displacement Response Calibration

spring stiffness phase. Finally, the adopted zone model with three zones of linear elastic springs as outlined in Fig. 12, shows excellent agreement with the experimental results. Since for the center nail, the center soil stiffness or confinement is of importance, the linear center soil stiffness of 34 MN/m^3 was plotted in Fig. 13 together with the measured pressure cell data in the center nail vicinity as well as with the original soil stiffness model used in the prediction. Very close agreement between the linear 34 MN/m^3 selected soil spring stiffness and the measured pressure cell data exists and can certainly be viewed as a very important validation of the developed PCYCO soil nail wall facing SSI model.

3.2 Analytical Parameter Studies

The objective of the parameter study was to identify the influence of parameter variations on the load-deflection behavior at the nail head. All parameter variations occurred within realistic prototype bounds of actual soil nail wall facing properties. In order to evaluate the influence of each individual variable, only one parameter at a time was adjusted and its influence on the soil nail facing behavior analyzed with respect to a common baseline case.

This baseline case assumed the dimensions, reinforcement ratios and design material properties, listed in Table 2, for a representative temporary shotcrete wall facing and a 1.53 m nail spacing. Parameters investigated can be grouped in five general categories of (1) soil, (2) facing panel, (3) bearing plate, (4) reinforcement, and (5) concrete properties.

Other parameters and constants used in the analytical model which were not subject to the parameter study reflect basic material characteristics for concrete and reinforcement as obtained from the tests. The concrete cracking strength f_{cr} was always taken as 10% of the nominal compression strength f'_c, the concrete modulus was evaluated following ACI 318 standard procedures.

Since for the soil structure interaction analysis the soil characteristics clearly have a dominant effect, four separate soil related parameters were investigated, namely the subgrade reaction coefficient related to the soil density, the load-deformation behavior of the soil approximated by a linear, bilinear or trilinear relationship, the diameter of the grout column around the nail from over-drilling or break-out of soil in the nail head vicinity, and finally a permanent cut-off soil pressure behind the wall facing panel. The influence of facing panel geometry, captured by the two parameters of facing panel thickness and soil nail spacing were also investigated. The steel bearing plate size was varied both in thickness and square side dimensions, in the form of a linear elastic overlay element in the plate region. Finally, variations in material quantities and properties were

evaluated by adding to the standard baseline 6 x 6, W 2.1 x 2.1 welded wire mesh, two waler bars in both directions ranging from #4 to #6 in reinforcement size.

Two different edge boundary conditions were investigated for each of the parameter studies, namely a free edge and a clamped edge condition with rotational constraints or fixity against rotations about the panel edges. The first or free boundary condition was the one used in the single nail experiments [5] and does not represent realistic field conditions except at the edges or corners of soil nail walls, whereas the clamped edge condition, again based on symmetry relationships, reflects prototype conditions in a continuous multi nail wall of infinite dimensions under uniform loading conditions.

TABLE 2. Baseline Panel Properties and Parameter Range

PARAMETER			BASELINE MODEL	PARAMETER RANGE
SOIL	Subgrade Reaction Coefficient	k_s	26.7 $\frac{MN}{in^3}$	15.7 - 37.6 $\frac{MN}{in^3}$
	Stiffness Variation	D_s	trilinear	linear, bilinear, trilinear
	Grout Column Diameter	D	203 mm	0 - 406 mm
	Permanent Soil Pressure	$P_{cut\text{-}off}$	0 kPa	0 - 28.7 kPa
FACING PANEL	Facing Thickness	t_{slab}	102 mm	102 - 203 mm
	Soil Nail Spacing	S_{nail}	1.53 m	1.22 - 1.83 m
	Finite Element Discretization	FE_{mesh}	M5	M4, M5, M6
BEARING PLATE	Plate Thickness	t_{plate}	19 mm	12.7 - 31.8 mm
	Plate Size	b_{plate}	203 x 203 mm	102 x 102 mm - 305 x 305 mm
REINFORCE -MENT	Welded Wire Fabric	w_{mesh}	6 x 6, W 2.1 x 2.1	none
	Waler Bars	#	2 # 4	2 # 4 - 2 # 6
CONCRETE	Design Compression Strength	f'_c	27.6 MPa	20.7 - 34.5 MPa

The above outlined parameters were investigated subsequently by individually varying one of them through the range indicated in Table 2, always starting with the reference baseline case. A total of eight groups of parameter studies were performed and are labeled in the following with A through H, representing:

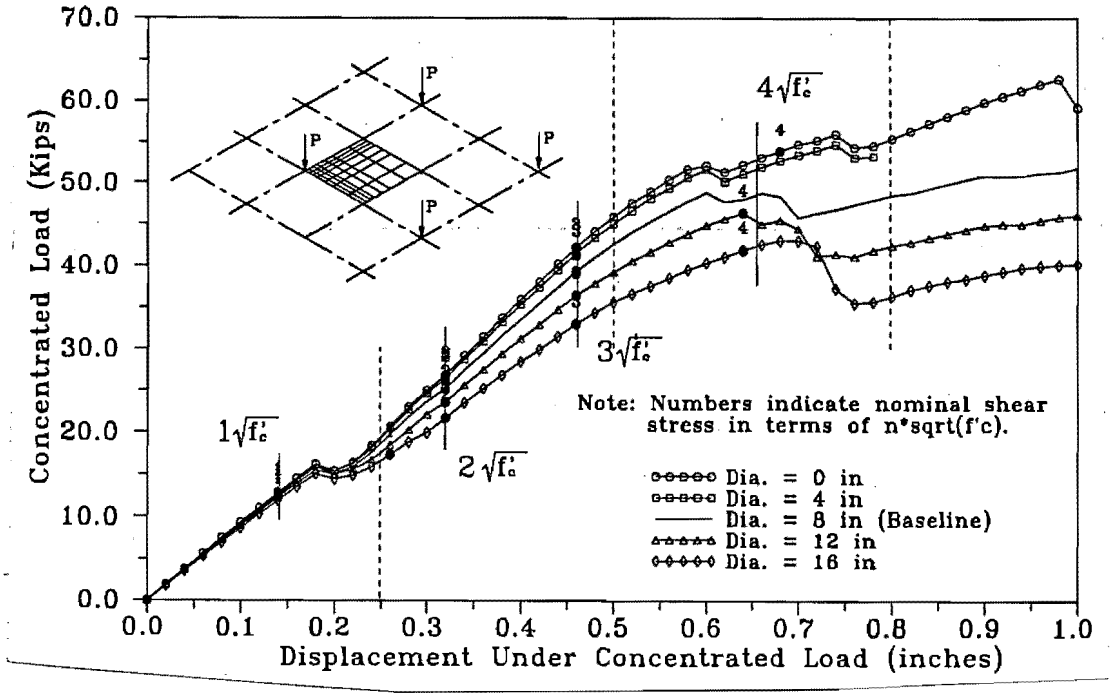
A: Grout Column Diameter Variations
B: Soil Stiffness Characterization
C: Permanent Cut-off Pressure Influence
D: Effects of Bearing Plate Dimensions
E: Moment Capacity or Reinforcement Ratio Variation
F: Facing Thickness Variation
G: Concrete Strength Influence
H: Soil Nail Spacing Variations

Results from the above parameter studies are graphically depicted in Fig. 22A-H.

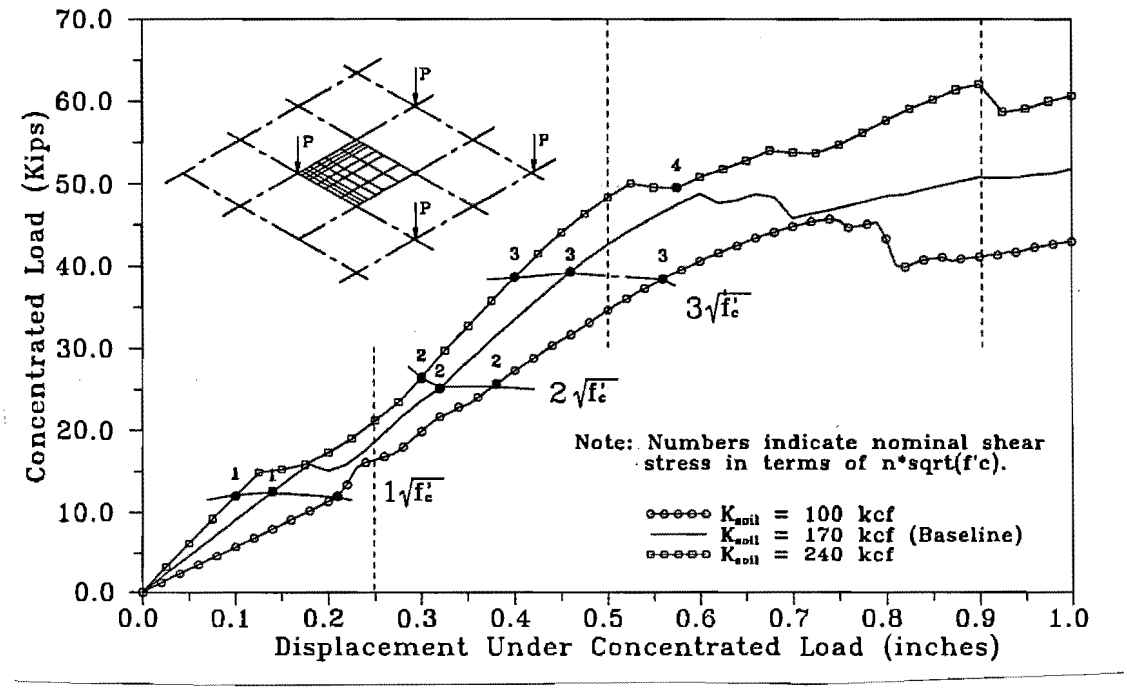
Parameter studies have shown that the nail load or facing capacity is most influenced by the facing panel thickness, by the bearing plate size, and the soil stiffness assumptions, whereas soil model variations, reinforcement ratios and soil nail spacings primarily affect the deformation states and not so much the nail loads. The evaluated shear stress indicators as multiples of $\sqrt{f_c'}$ showed the largest dependence on the assumed grout column diameter and on the actual concrete compression strength. Single nail tests showed that punching shear failures are possible when the shear indicators exceed the parameter studies clearly indicate for which of the investigated parameter variations and parameter combinations this punching shear failure scenario becomes more likely. For example, low concrete strength combined with a large grout column can quickly result in a punching shear failure mode. However, both test and analyses clearly showed that punching shear does only occur at large deformation states and is preceded by significant inelastic flexural action in the form of cracking, yielding and large relative facing panel deformations which will provide significant warning prior to punching shear failure at least in thin temporary shotcrete facings.

3.3 Analytical Case Studies

The same analytical model, with the exception of linear tension gap springs instead of trilinear springs, was employed for case studies on representative soil nail wall facing designs which are presented in the following. In addition to the twelve case studies, the effects of different reinforcement ratios, both uniformly distributed and concentrated along nail head strips, on the nail

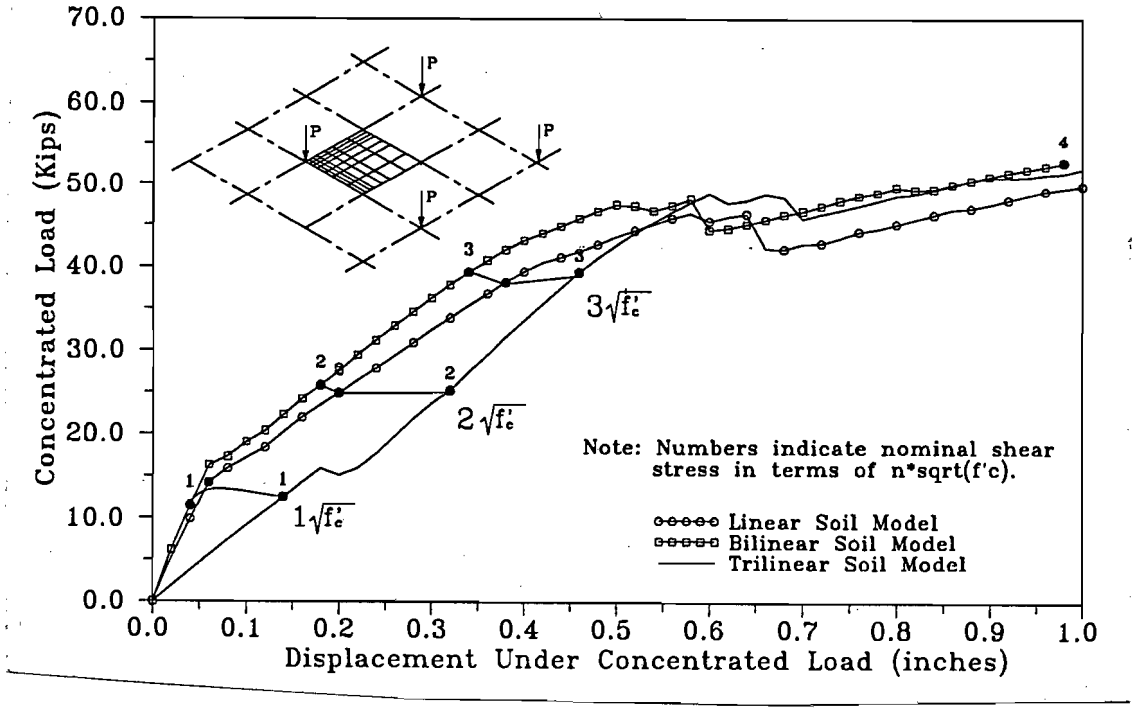


a) Grout Column Parameter Study (Multi Nail), A

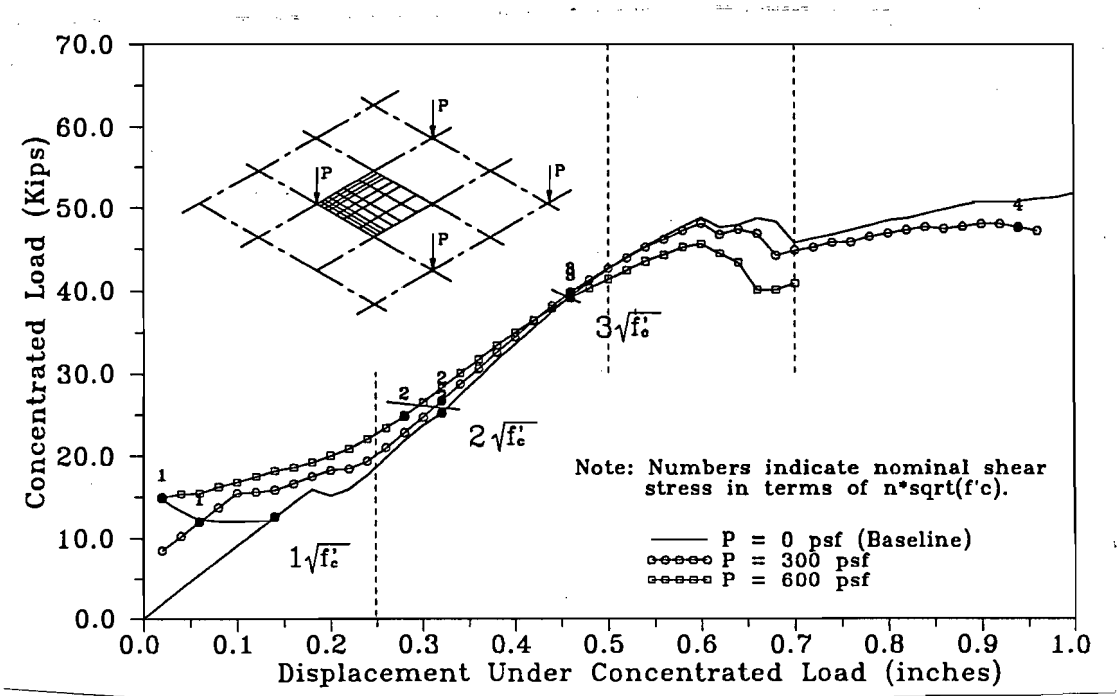


b1) Soil Stiffness Parameter Study, B1

FIG 22. Soil Nail Facing Parameter Study

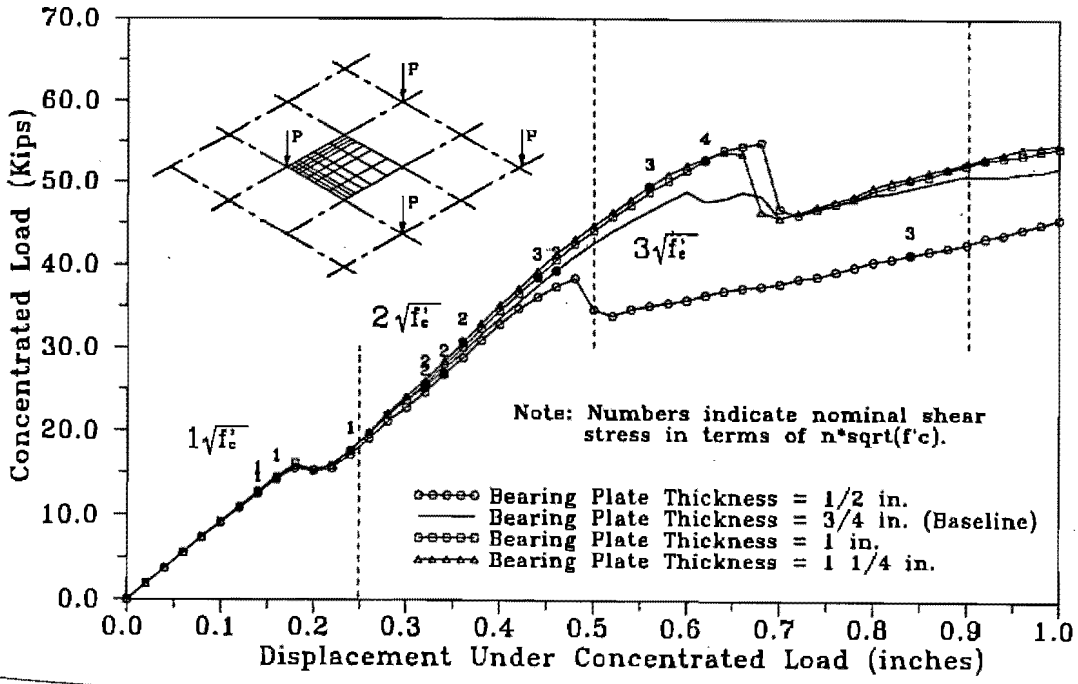


b2) Soil Model Parameter Study, B2

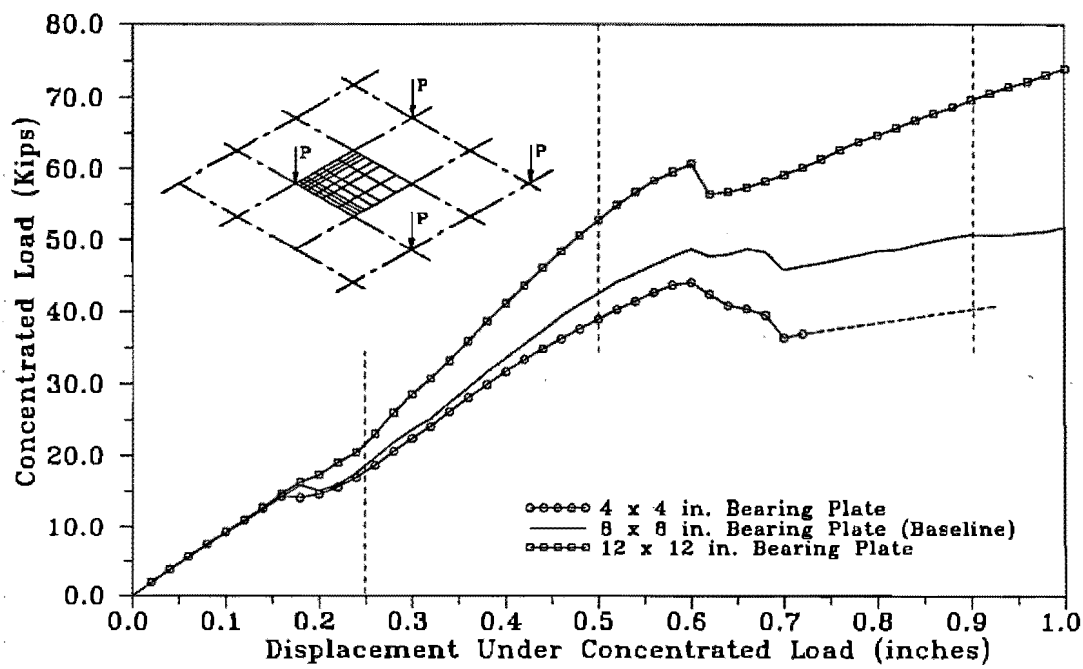


c) Soil Cut-off Pressure Parameter Study, C

FIG 22. Soil Nail Facing Parameter Study (continued)

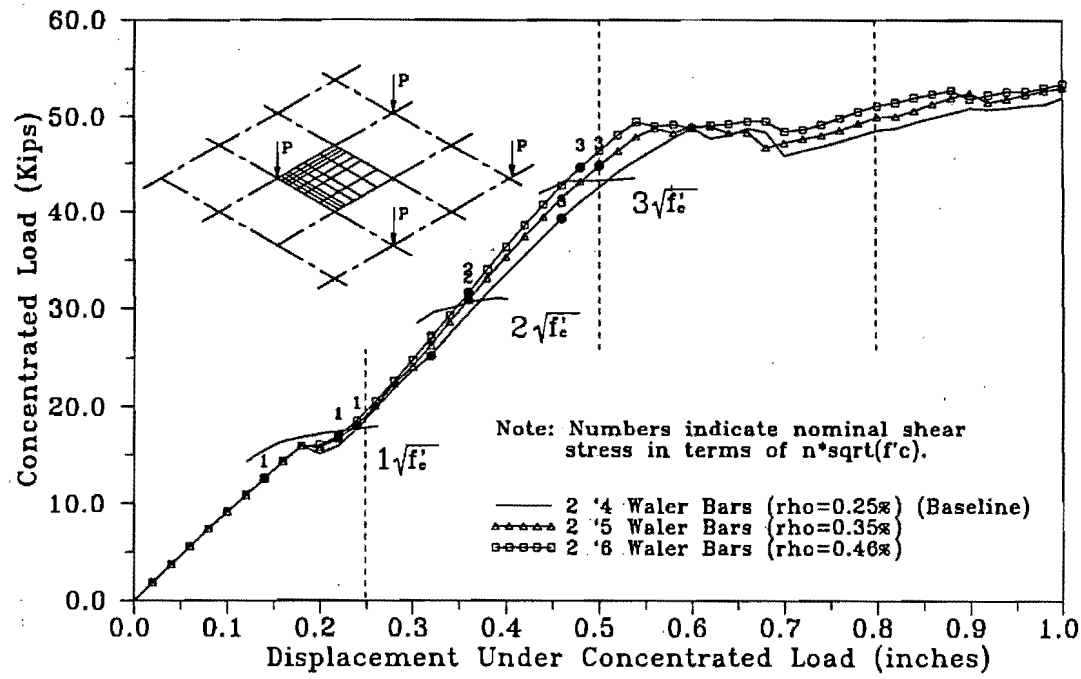


d1) Bearing Plate Thickness Parameter Study, D1

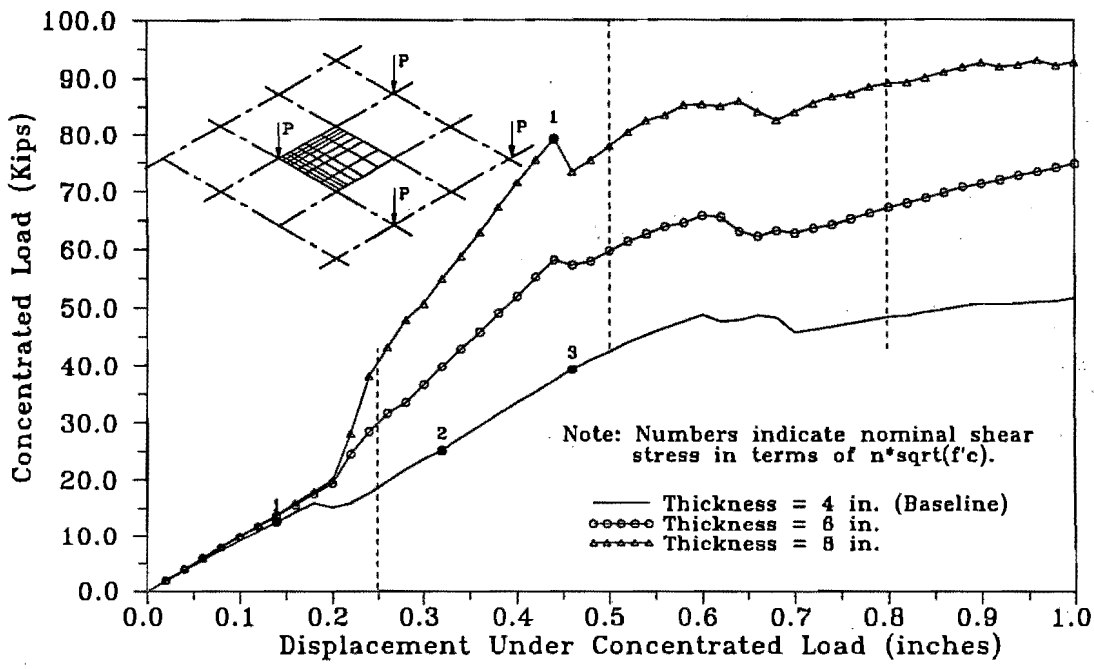


d2) Bearing Plate Size Parameter Study, D2

FIG 22. Soil Nail Facing Parameter Study (continued)

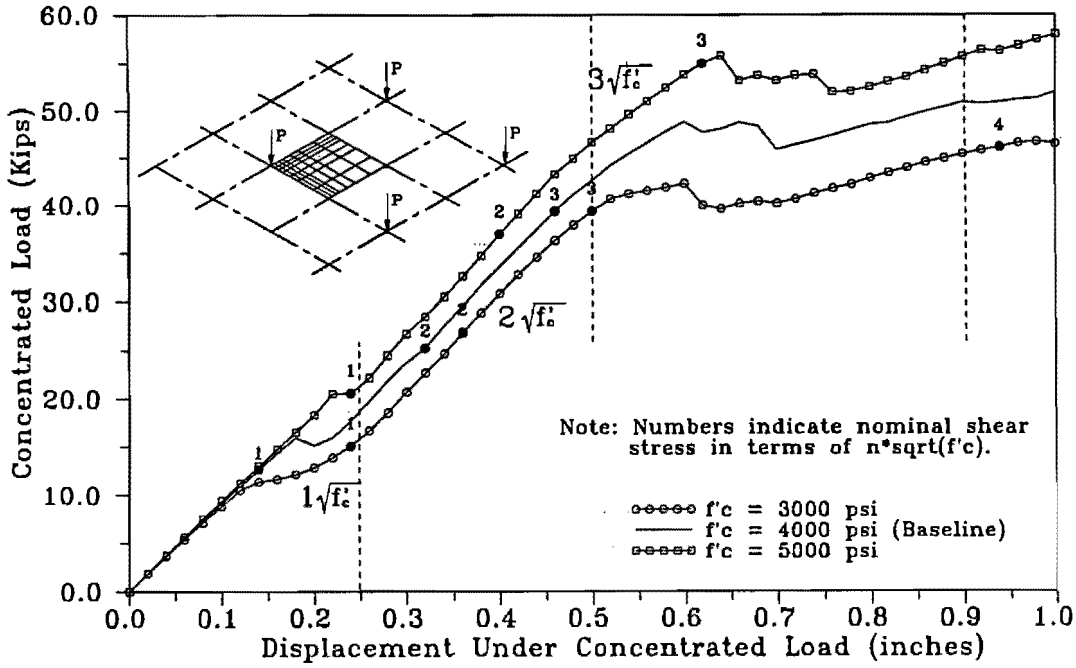


e) Yield Moment Parameter Study, E

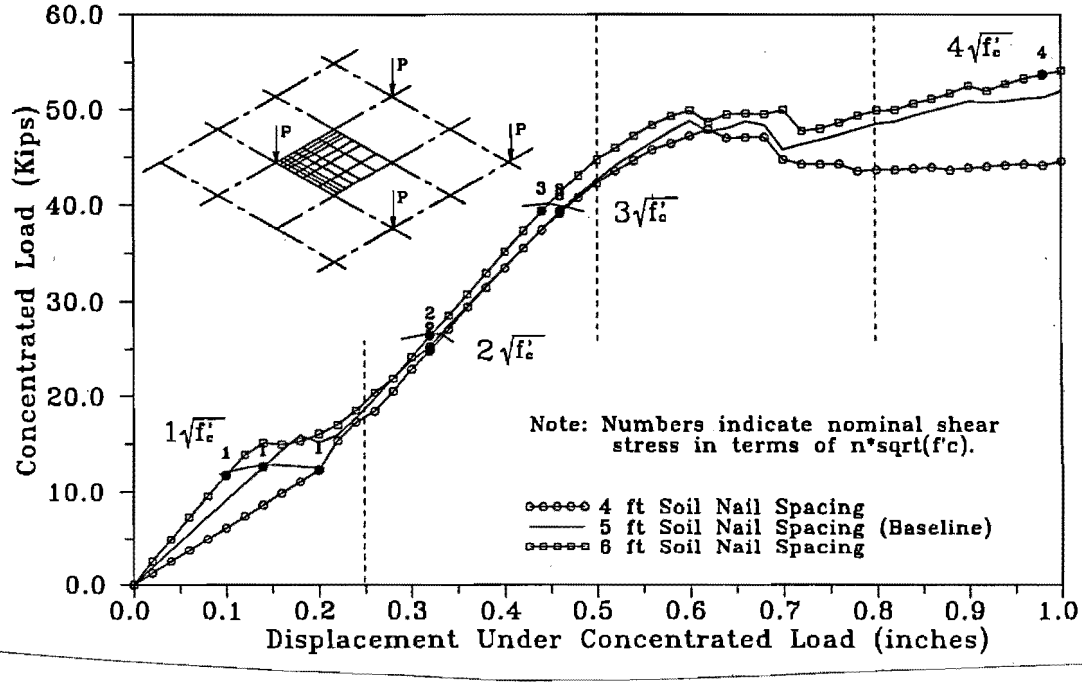


f) Slab Thickness Parameter Study, F

FIG 22. Soil Nail Facing Parameter Study (continued)



g) **Concrete Strength Parameter Study, G**



h) **Nail Spacing Pressure Parameter Study, H**

FIG 22. Soil Nail Facing Parameter Study (continued)

head load-deformation response were investigated to demonstrate the complex interaction between facing stiffness/capacity and the Winkler spring simulation of the soil loading.

A total of twelve case studies of representative soil nail wall facing designs were investigated, half of them for typical 102 mm temporary shotcrete facings and the other half for 203 mm permanent shotcrete or cast-in-place wall facings. All analyses were based on a 1.83 m typical nail head module in both directions, with other constant design parameters listed in Table 3.

The six temporary shotcrete facing cases had nail head plates of 203 mm square dimensions with a 19 mm plate thickness, effective soil reaction moduli varying between 9.4 MN/m^3 and 37.6 MN/m^3 and total reinforcement ratios referenced to the 1.83 m x 102 mm facing cross-section of ρ = 0.222% and 0.465%, including both uniformly distributed welded wire fabric and waler bars in the nail head strips.

The 203 mm thick permanent shotcrete or cast-in-place wall facing case studies were based in addition to the constant case parameters in Table 3 on 229 mm square and 25 mm thick nail head plates. Reinforcement for the permanent facings consisted of 3 waler bars along the nail head strips and uniformly distributed mild reinforcement on a 305 mm spacing in both directions with overall reinforcement ratios of ρ = 0.313% and 0.484%.

A summary of all twelve case study parameter variations is provided in the case study parameter matrix in Table 4. Results from the individual case studies are summarized in the following, in the form of nail head load versus nail head deflection relationships. All analyses were performed for continuous facing panels with the 1.83 m nail spacings in both directions.

TABLE 3. Constant Reference Case Parameters

Facing Materials	Concrete Properties f_c'	27.6 MPa
	Steel Properties { waler bars f_y	455 MPa
	Steel Properties { wire fabric f_y	517 MPa
Soil Properties	Cutoff Pressure, P_C	28.7 kN/m^2
	Grout Column Diameter, D_{GC}	203 mm
Nail Head Parameters	Headed-Stud Spacing, S_{HS}	152 mm
	Headed-Stud Body Diameter, d_{HS}	19 mm
	Headed-Stud Head Diameter, d_H	32 mm
	Cover to Headed Stud, C_{HS}	51 mm
	Cover to Plate, C_{PL}	0 mm
	Nail Spacing S	152 mm

TABLE 4. Case Study Parameter Matrix

Case	Facing Thickness h_P (mm)	Plate Width b_{PL} (mm)	Plate Thickness t_{PL} (mm)	Soil Effective k_S (MN/m^3)	Waler Steel	Mesh/Mat Steel	Reinforce-ment Ratio ρ (%)
1	102	203	19	9.4	2-#4	W2.0-6x6	
2	102	203	19	18.8	2-#4	W2.0-6x6	0.222
3	102	203	19	37.6	2-#4	W2.0-6x6	
4	102	203	19	9.4	2-#5	W4.0-4x4	
5	102	203	19	18.8	2-#5	W4.0-4x4	0.465
6	102	203	19	37.6	2-#5	W4.0-4x4	
7	203	229	25	9.4	3-#4	#4 @ 12"	
8	203	229	25	18.8	3-#4	#4 @ 12"	0.313
9	203	229	25	37.6	3-#4	#4 @ 12"	
10	203	229	25	9.4	3-#5	#5 @ 12"	
11	203	229	25	18.8	3-#5	#5 @ 12"	0.484
12	203	229	25	37.6	3-#5	#5 @ 12"	

The load-deflection response for the temporary shotcrete facing panels with 102 mm thickness is depicted in Fig. 23 for different reinforcing patterns and soil reaction coefficients. As can be seen from the response analyses in Fig. 23, the soil reaction coefficient has a significant influence on the nail head/facing capacity. Furthermore, the facing panel capacity clearly seems to increase with increasing reinforcement ratios which is somewhat contradictory to the findings in the initial analytical parameter study [7]. However, more than doubling the overall reinforcement ratio from 0.222% to 0.465% increases the nail load capacity at a nail head displacement of 20.3 mm by only 20% to 30% for the different soil reaction coefficients, which is still significantly less than would be expected from the simplified assumption that a doubling of the reinforcement amount should also approximately double the facing capacity.

The load-deflection response characteristics obtained from the permanent facing case studies are depicted in Fig. 24 for reinforcement ratios of ρ = 0.313% and 0.484%, respectively. Again, the influence of the soil reaction coefficient was very pronounced, particularly at the lower deformation limit states, i.e. nail head displacements of 10.2 mm or less, whereas subsequent to the onset of reinforcement yielding and first concrete crushing this influence of the soil reaction coefficient seemed to diminish.

Increases in the reinforcement ratio by a factor of 0.484/0.313 = 1.55 resulted in nail head capacity increases at the 203 mm nail head displacement limit state of only 7.5% to 15% with the larger increases for the lower soil reaction coefficients. Again, this capacity was less than expected based on the increase in overall reinforcement ratio.

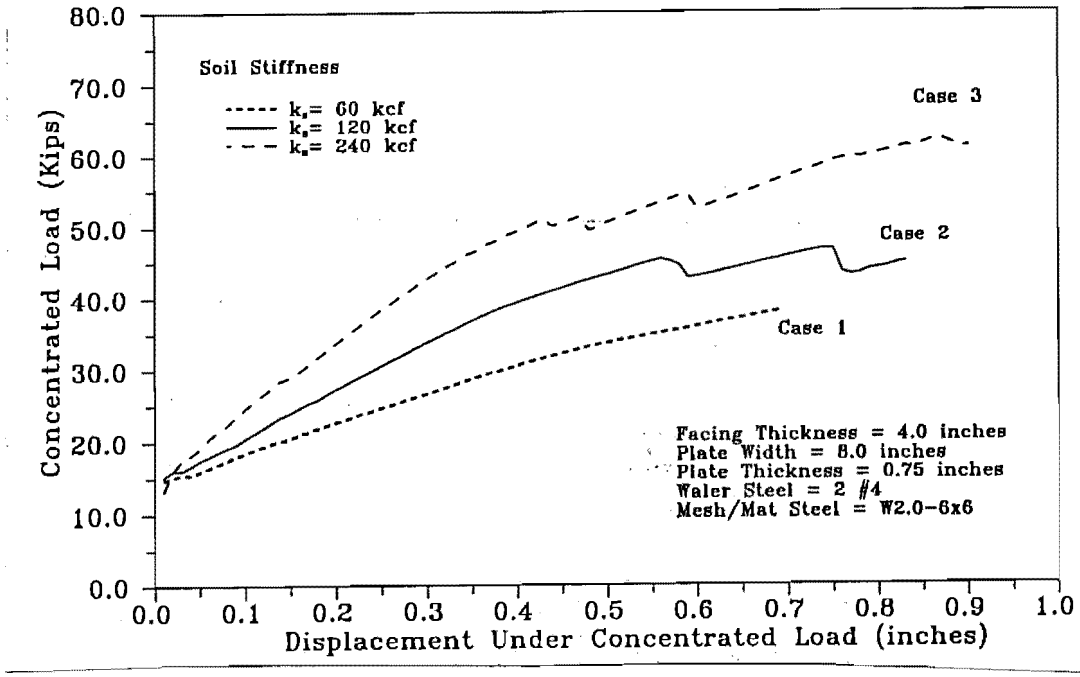
To investigate the reasons for this observed behavior further, additional parametric studies were performed where the parameter variations consisted of the reinforcement ratio and the reinforcement distribution as will be explained in the following.

The first set of parameter studies into the influence of the facing reinforcing patterns addressed a baseline case of a temporary 102 mm shotcrete facing with a W 2.0 – 6 x 6 mesh, and a soil reaction coefficient of 37.6 MN/m^3; all other variables are as defined in Tables 3 and 4. The parameter variations consisted of 2 waler bars in each direction in the form of 2 #3, 2 #4, 2 #5, and 2 #6 grade 60 bars, resulting in overall reinforcement ratios between 0.16% and 0.39%.

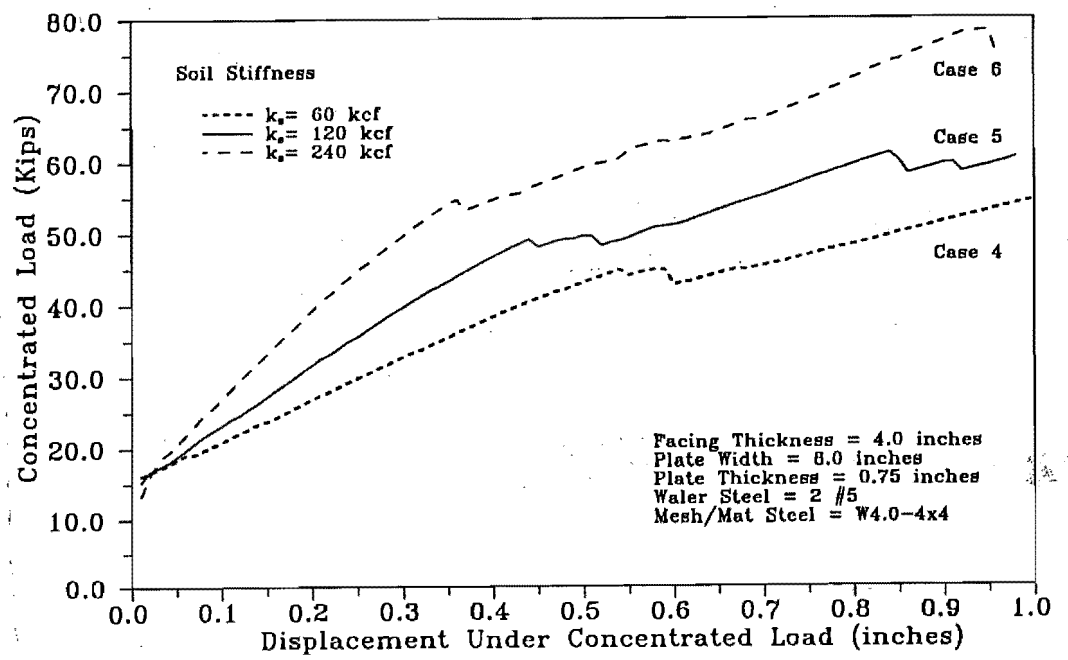
The resulting load-deflection response at the nail head is depicted in Fig. 25a for all four cases and shows that reinforcement amount changes in the form of waler bars or strip reinforcement have very little effect on the load-deformation characteristics of the facing panels. At the 20 mm nail head displacement limit state, only a 10% increase in nail head capacity can be observed over the depicted reinforcement ratio range.

Using, instead of concentrated reinforcement along the nail head lines in the form of waler bars, only distributed reinforcement amount variations as depicted in Fig. 25b, a wider range of load-deflection response curves is obtained with an increase of around 20% in load capacity at the 20 mm nail head displacement limit state over the same reinforcement ratio range of 0.15% to 0.4%. Thus, the influence of the reinforcement distribution had a greater effect on the facing capacity than the reinforcement amount. The same findings were obtained for similar case studies in 8 in. (203 mm) permanent facing panels.

Explanations for the observed behavior can be found in the facing panel deformation profiles and the interaction between the facing panels and the soil reaction springs. The soil spring stiffnesses and the reinforcement distribution together with the resulting deformation profile changes have a significantly larger influence on the nail head facing capacities than the overall reinforcement amounts.

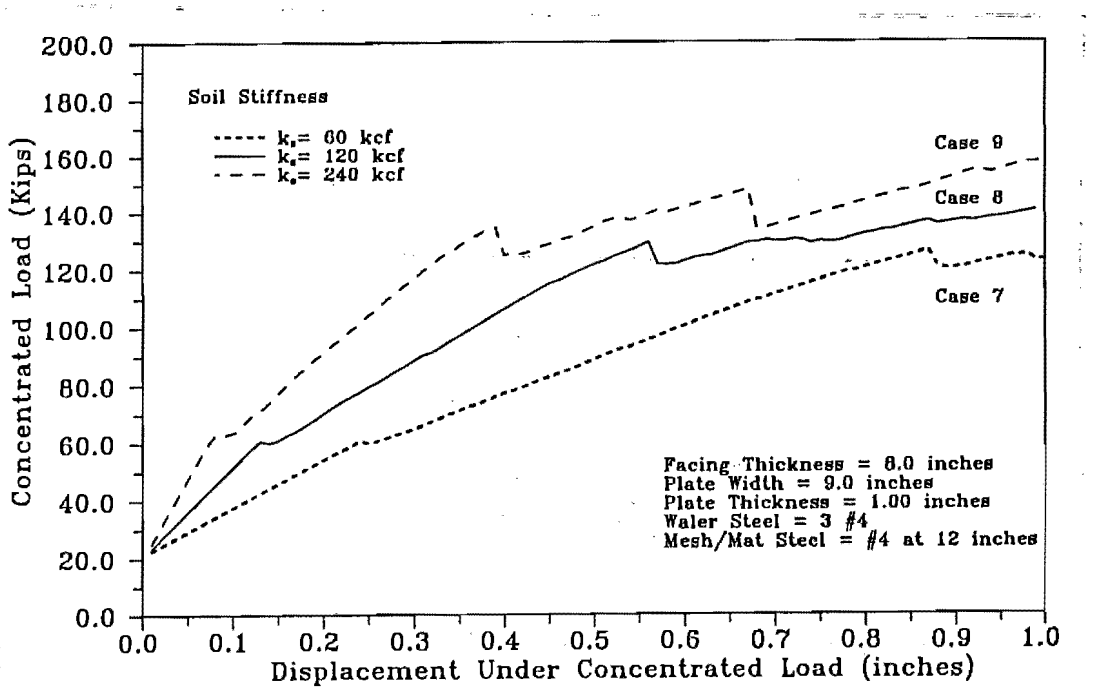


a) Nail Head Load-Deflection Behavior, Cases 1 to 3

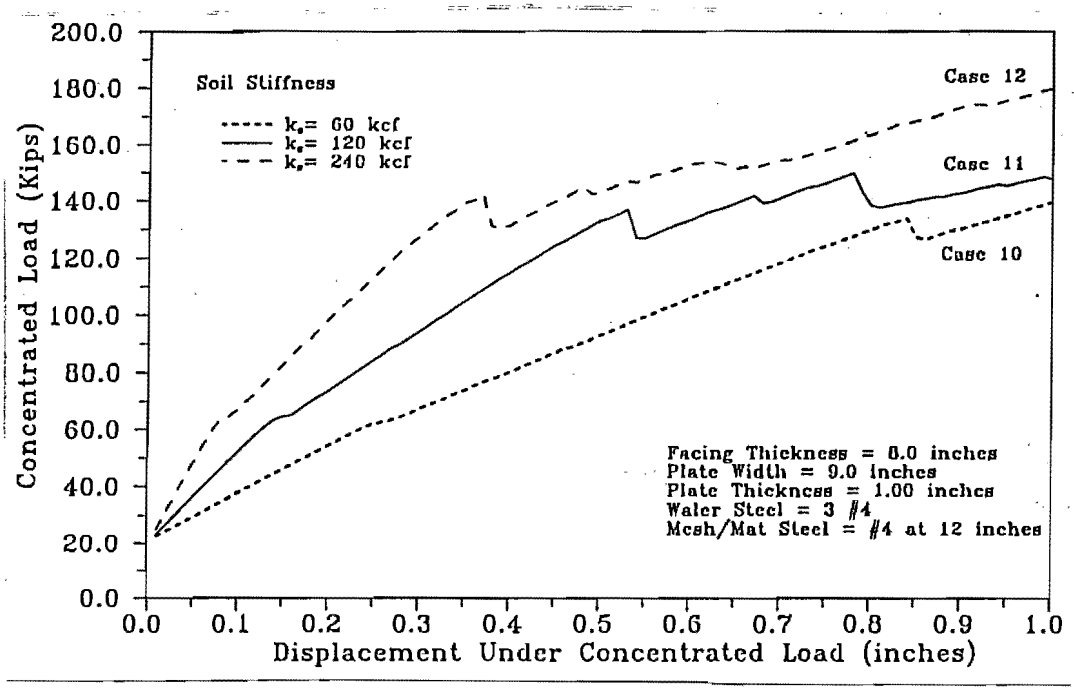


b) Nail Head Load-Deflection Behavior, Cases 4 to 6

FIG 23. Temporary Facing Panel Case Studies

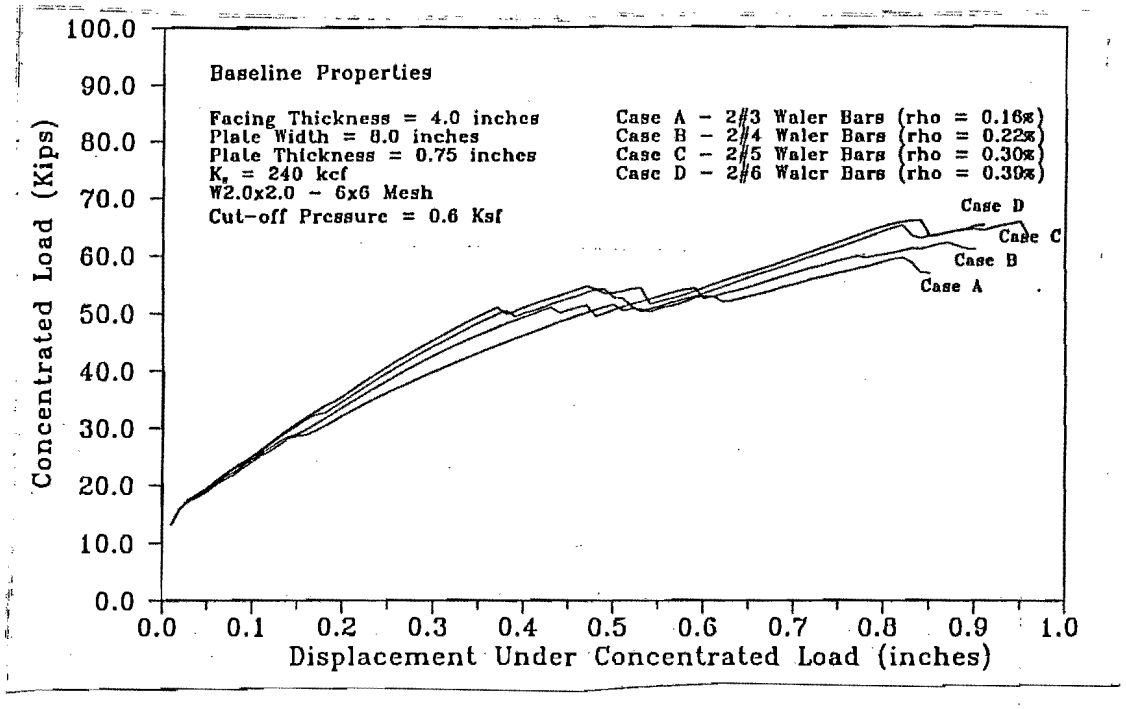


a) Nail Head Load-Deflection Behavior, Cases 7 to 9

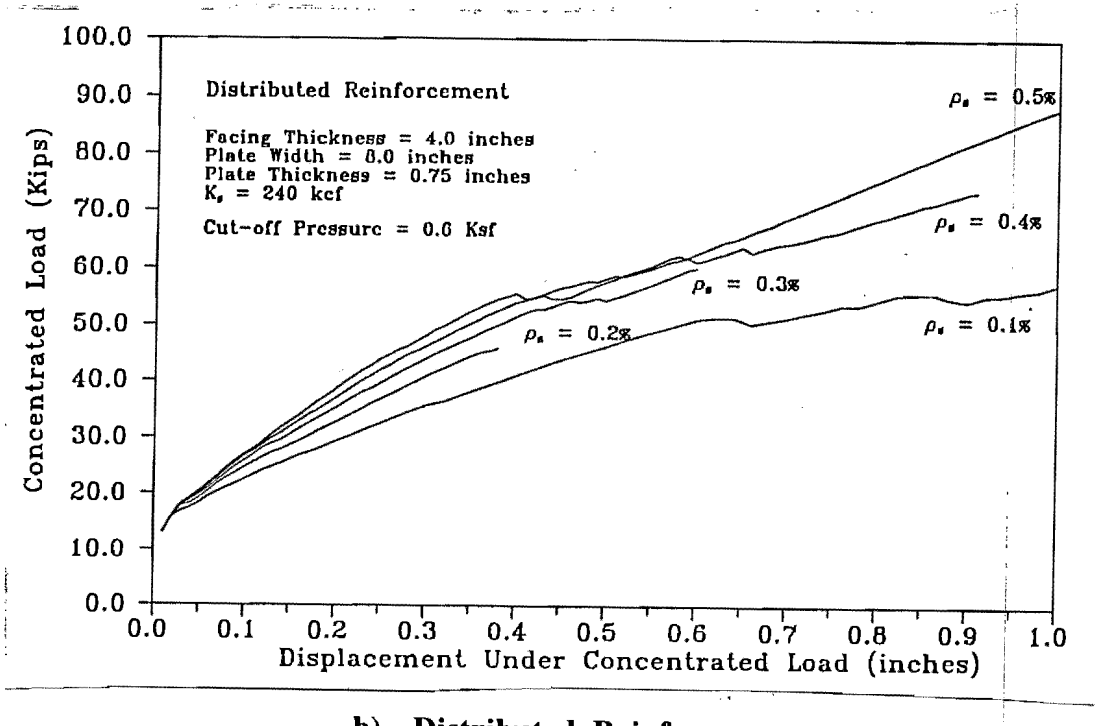


b) Nail Head Load-Deflection Behavior, Cases 10 to 12

FIG 24. Permanent Facing Panel Case Studies



a) Strip Reinforcement



b) Distributed Reinforcement

FIG 25. Reinforcement Ratio Effect on Nail Head Load-Deflection Behavior

4. SUMMARY AND CONCLUSIONS

In direct support of the FHWA demonstration project DP-103 and the development of comprehensive and rational limit state design models for soil nail walls, the influence of parameter variations on the response behavior of soil nail wall facing panels was investigated.

The parameter studies were performed with a specially developed nonlinear finite element model which can capture material nonlinearities in the shotcrete or cast-in-place soil nail wall facing as well as in the supporting soil reactions, thus, modeling the complex nonlinear soil structure interaction.

The analytical model was calibrated with test results from six full-scale single nail tests conducted in a specially designed and constructed soil box and verified by means of a full-scale nine nail facing panel test. The model validation was achieved with direct measurements of the soil pressure under the facing panel and a one-on-one comparison with pressure or soil reaction stiffness values used in the analytical model. Results from both the analytical model and the full-scale experiments showed that for typical temporary shotcrete facing panels, very ductile failure modes can be expected due to membrane action in the reinforcement and direct soil support under potential punching shear cones. Thus, failure mechanisms with significant warning through large deformations prior to capacity loss are likely. Only in overlayed permanent walls this large deformation warning is lost since punching and delamination of the temporary facing panel can occur behind the permanent wall facing without any visible signs of distress. The analytical parameter studies furthermore showed that the nail load or facing capacity was most influenced by the facing panel thickness, the bearing plate size, and the soil stiffness assumptions, whereas reinforcement ratios and nail spacings primarily affected the deformation states to the highly coupled soil structure interaction effect.

Case studies with common design details encountered in soil nail wall projects reconfirmed these findings and can now be used as the basis for detailed design model developments in direct support of FHWA demonstration project DP-103.

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