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STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING SERVICES
OFFICE OF TRANSPORTATION LABORATORY

# SOIL PRESSURE AND STRESSES ON WOOD FACED RETAINING WALL WITH BAR-MAT ANCHORS

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## CONVERSION FACTORS

### English to Metric System (SI) of Measurement

| Quantity | English unit | Multiply by | To get metric equivalent |
|---|---|---|---|
| Length | inches (in)or(") | 25.40<br>.02540 | millimetres (mmm)<br>metres (m) |
| | feet (ft)or(') | .3048 | metres (m) |
| | miles (mi) | 1.609 | kilometres (km) |
| Area | square inches ($in^2$) | $6.432 \times 10^{-4}$ | square metres ($m^2$) |
| | square feet ($ft^2$)<br>acres | .09290<br>.4047 | square metres ($m^2$)<br>hectares (ha) |
| Volume | gallons (gal) | 3.785 | litres (l) |
| | cubic feet ($ft^3$) | .02832 | cubic metres ($m^3$) |
| | cubic yards ($yd^3$) | .7646 | cubic metres ($m^3$) |
| Volume/Time (Flow) | cubic feet per second ($ft^3/s$) | 28.317 | litres per second (l/s) |
| | gallons per minute (gal/min) | .06309 | litres per second (l/s) |
| Mass | pounds (lb) | .4536 | kilograms (kg) |
| Velocity | miles per hour (mph)<br>feet per second (fps) | .4470<br>.3048 | metres per second (m/s)<br>metres per second (m/s) |
| Acceleration | feet per second squared ($ft/s^2$) | .3048 | metres per second squared ($m/s^2$) |
| | acceleration due to force of gravity (G) | 9.807 | metres per second squared ($m/s^2$) |
| Weight Density | pounds per cubic ($lb/ft^3$) | 16.02 | kilograms per cubic metre ($kg/m^3$) |
| Force | pounds (lbs)<br>kips (1000 lbs) | 4.448<br>4448 | newtons (N)<br>newtons (N) |
| Thermal Energy | British thermal unit (BTU) | 1055 | joules (J) |
| Mechanical Energy | foot-pounds (ft-lb)<br>foot-kips (ft-k) | 1.356<br>1356 | joules (J)<br>joules (J) |
| Bending Moment or Torque | inch-pounds (ft-lbs)<br>foot-pounds (ft-lbs) | .1130<br>1.356 | newton-metres (Nm)<br>newton-metres (Nm) |
| Pressure | pounds per square inch (psi) | 6895 | pascals (Pa) |
| | pounds per square foot (psf) | 47.88 | pascals (Pa) |
| Stress Intensity | kips per square inch square root inch (ksi $\sqrt{in}$) | 1.0988 | mega pascals $\sqrt{metre}$ (MPa $\sqrt{m}$) |
| | pounds per square inch square root inch (psi $\sqrt{in}$) | 1.0988 | kilo pascals $\sqrt{metre}$ (KPa $\sqrt{m}$) |
| Plane Angle | degrees (°) | 0.0175 | radians (rad) |
| Temperature | degrees fahrenheit (F) | $\frac{tF - 32}{1.8} = tC$ | degrees celsius (°C) |

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# I. INTRODUCTION

The lateral earth pressure that will be exerted by soil on the back face of a retaining wall is a major governing factor in its design. Based on geometric configuration, earth retaining systems have generally been classified as either gravity walls, cantilever walls, or counterfort walls. In recent years, several new types of earth retaining systems have been developed which utilize the strength and weight of the backfill material to achieve internal stability. These new types of earth structures include the proprietary system of Reinforced Earth (Vidal, 1969; Chang, 1974), bar-mat anchored system of mechanically stabilized embankment (MSE) (Chang et al, 1981) and tire anchored timber walls (Jackura et al, 1983). The facing members for these earth retaining systems can be of either precast reinforced concrete, steel, or wood. In all cases, it is necessary to evaluate the earth pressure that the structure must withstand.

The objectives of the study reported herein were to measure both the lateral soil pressure and the stresses developed in the plywood facing and steel bar-mats of an experimental anchored bulkhead (MSE), using field instrumentation to develop realistic design criteria for this system under varying soil conditions.

The determination of lateral soil pressure on concrete retaining walls is generally based on Rankine's State of Stress Theory (1857) or Coulomb's Sliding Wedge Theory (1776), in which lateral pressure is directly proportional to the vertical soil stress (or the weight of the soil above the point of concern). Thus, practical engineers often assume an equivalent fluid pressure for design purposes. The distribution of lateral pressure with depth, therefore, is assumed to be triangular. For internally braced, flexible walls, Terzaghi and Peck (Terzaghi, 1968) assumed a uniform distribution of lateral soil pressure for sands and trapezoidal distribution for soft to medium clays and stiff clays. Many practitioners have applied empirical rules developed for internally braced walls to the design of tied-back walls (Goldberg, 1976). At the present time, no empirical methods for tied-back walls have been accepted as universally as Peck's criteria for internally braced, flexible walls.

Various investigators have developed charts and tables to aid in determining the magnitude of lateral earth pressures for given conditions (Tschebotarioff, 1951). However, in many instances, the earth pressure is estimated with the aid of equations developed from the classical theories derived either by Rankine (1857) or Coulomb (1776), by which the lateral earth pressure is assumed to be equal to the vertical stress, or the weight of a unit column of soil at the point of concern multiplied by a coefficient of lateral earth pressure. The coefficient is a function of wall movement, physical properties of the backfill, and geometry of the wall and the backfill. Recently, Coyle et al (1974) and Prescott et al (1973), reported their research results on lateral earth pressure on concrete-faced retaining walls. Schuster et al (1973), conducted a comprehensive literature review of earth pressure on timber crib walls. They also utilized instrumentation on a prototype timber crib wall to verify the bin pressure equations developed by Airy (1898) and the modified Janssen's equation (Jumikis, 1971). The bin pressure is generally utilized for design of crib wall members. Schuster et al (1974), also instrumented the timber members with strain gages to determine the stresses developed in the stretchers and the headers. The results of these field instrumentation studies provided only limited information on the soil pressure and deformation of the timber members. The Transportation Laboratory (TransLab) of the California Department of Transportation (Caltrans) instrumented a reinforced earth wall and two MSE walls on Interstate Highway 5 at Dunsmuir to study the lateral earth pressure on precast concrete facing members. The results were reported in 1981 by Chang, Hannon and Forsyth. A second report by Hannon and Forsyth (1984) also discusses lateral earth pressure measured on two concretefaced MSE walls constructed with low quality backfill. Testing of steel face members was reported by Chang (1974) for the first reinforced earth wall constructed in the United States on California Highway 39 in Los Angeles County.

This report presents the results of both laboratory testing and field instrumentation of a wood-faced mechanically stabilized embankment wall (WFMSE) located on Highway 99 at Delhi, California (Figure 1). The wall was built in April, 1979 as a temporary retaining structure to permit use of the existing northbound roadway while a partial embankment was constructed over the existing southbound roadway for the new highway alignment. After completion of the embankment, a new roadway was constructed behind the wall and traffic was transferred from the original highway. The outer side slope of the embankment for the northbound lanes was placed against the wall which was left in place with only the top panels of the wall removed. The completed embankment encapsulated the temporary wall and covered the original highway on an elevated grade through the town of Delhi.

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## II. CONCLUSIONS

The WFMSE satisfactorily performed its function as a temporary retaining wall. It was cost-efficient and easily constructed within 18 working days. Following are more specific conclusions based on observations and measurements:

### A. Lateral Soil Pressure

1. The laboratory vacuum test on the plywood panels proved to be a good indicator for predicting lateral soil pressure in the field.

2. An excellent correlation existed between calculated and estimated lateral soil pressures from horizontal strain gage measurements on the face panels during construction and during laboratory vacuum testing.

3. A reasonable correlation was found between lateral soil pressure derived from panel stresses and bar-mat stresses near the wall face.

4. Due to possible movement at the soil-pressure cell interface as a result of warpage of the plywood panels, soil pressure cells provided somewhat unreliable readings during the latter stages of wall construction. When analyzed alone, the pressure cells did not provide a good indicator of soil pressure distribution behind the wall face.

5. When lateral soil pressure data from pressure cells, actual plywood panel strains, laboratory vacuum tests, and bar-mat stresses are combined, the resulting lateral soil pressure diagram followed a rectangular (uniform) distribution behind the wall face.

B. <u>Bolts and Bar-Mats</u>

1. Bar-mat stresses peaked after completion of the wall, relaxed somewhat, then increased until the roadway was completed.

2. The maximum recorded bar-mat stress was 19.5 ksi in Levels 2 and 3, seven feet back from the wall face prior to encapsulation within the embankment during October 1979.

3. Substantial bending was experienced by the anchorage (connector) bolts, possibly due to a deficiency of soil below the bar-mats at the wood-soil interface or possible settlement of the soil mass relative to the wall face.

4. The strain gages on the connector bolts provided questionable results since all transfer connection members were bolted flush against the back of the wood panel facing members.

C. <u>Douglas Fir Posts</u>

1. Analysis of strain gage data on the instrumented vertical member denotes compression on the outer face and tension on the inner face (plywood side) as a result of outward bending. The maximum measured bending stress was 1002 psi, well below the allowable stress of about 2000 psi.

2. The instrumented vertical post was not spliced and, therefore, did not truly represent the response of the vertical members.

3. The difference in stiffness of the spliced and continuous vertical posts influenced the stresses that were measured in the field.

4. Continuous vertical posts for the full length of wall would have maintained better wall alignment during construction loading, and the tilting or leaning evident in the upper portion of wall could have been avoided.

D. Plywood Panels

1. The strain gages on the plywood face panels provided reliable data through completion of the wall on May 2, 1979. Strain gage readings increased significantly after wall construction with the highest readings recorded on the inside face of wall.

2. Strain gages on the plywood face may have experienced creep or partial penetration of moisture which led to malfunction. The strain gages on the outside wall face were more stable than the strain gages mounted at the inside wall/soil interface.

# III. RECOMMENDATIONS

More utilization should be made of WFMSE systems to reduce the cost of retaining wall construction of both temporary and permanent facilities. Efforts should be made to take full economic advantage of MSE by utilizing available on-site materials in lieu of importing more costly, higher quality backfill material. The design for internal stability should also be predicted on the redistribution of stresses in the backfill. When feasible, based on internal and external stability, a reduction in the bar-mat embedment length in the upper portion of MSE walls should also be considered for potential cost savings. The following are specific recommendations relative to WFMSE systems based on this project:

## A. Bolts and Bar-Mats

1. Backfill material placed over the mats should be compacted starting from the wall and proceeding back from the face to avoid "bunching" of the mats near the wall. Hand guided compaction equipment should be operated near the wall face.

2. On future instrumented installations, strain readings of bolts should be monitored before and after tightening to confirm validity of measurements and the torque should be specified to ensure a uniform initial level throughout the wall during construction.

3. The compacted fill height should be brought up to a point directly beneath the mat elevations prior to mat placement to eliminate excess bending moments in the bar-mat connection bolts.

B. Wood Facing

1. Full length vertical posts should be utilized when practical.

2. To minimize the effects of moisture on strain readings, all future instrumented wood members (posts or panels) should be treated to maintain moisture content.

3. The flexible nature of the wall should be given greater consideration during construction to avoid undue deformation of the wall facing.

4. Consideration should be given to constructing the wall face on a slight batter so that subsequent outward movement would rotate the wall to a more nearly vertical position. This would be especially important on permanent installations.

C. Soil Instrumentation

The problems encountered in obtaining lateral soil pressure data from the pressure cells should be investigated and corrected on future installations. Since plywood is flexible and susceptible to moisture penetration and surface deformation, the installation of lateral soil pressure cells is questionable. Even with moisture sealing of the panels, movement and stress concentrations could result in erroneous readings. Therefore, to attain realistic readings of soil pressure at the face, soil pressure cells should be mounted to minimize these effects.

Future WFMSE studies should consider long-term performance of both the wood-facing and the bar-mats. These walls should be designed with a rectangular (uniform) pressure distribution when used with fine grained cohesionless backfill.

# IV. IMPLEMENTATION

Mechanically stabilized embankments (MSE) with wood or precast concrete facing are presently recommended for utilization by Caltrans whenever feasible as an alternative to other types of retaining wall construction. MSE is a Caltrans development and is licensed through a joint agreement under the Reinforced Earth (RE) Company patent for use on Caltrans projects. Under the agreement, RE is given the opportunity to submit an alternative design on all projects where MSE is proposed.

Presently, TransLab performs feasibility studies for both MSE and RE systems at district request. Final MSE design is performed by the Office of Structures Design. Some designs will be performed by districts. Experience has shown that these systems have provided satisfactory performance and significant construction cost savings compared to crib wall and other retaining wall construction.

# V. DISCUSSION

## A. General Description of the Construction Project

Construction of a vertical earth retaining wall at Delhi, California was necessary in order to temporarily support the embankment fill of a new elevated six-lane freeway and prevent it from encroaching on the existing northbound lanes of Road 10-Mer-99 during construction. Northbound traffic was carried through construction on the existing northbound lanes and southbound traffic moved through the project on the new embankment construction. The new freeway on fill reached a height of 24 feet (Figure 2). The right-of-way was limited so that detouring of the ongoing traffic was not possible.

This bulkhead-type retaining wall of approximately 1300-foot length was required only while a partial embankment for the new freeway was being constructed. After completion, the fill slope over the existing northbound lanes would be constructed to the level of the new roadway (refer to Figure 3). The existing road was obliterated through this process.

A temporary MSE wall was conceived for this purpose. Soil reinforcing systems, such as the patented concept of Reinforced Earth (RE) which utilizes steel strips as reinforcement or MSE (steel bar-mats for reinforcement), have proven to be more economical for this type of application than standard concrete retaining walls or crib walls of steel or concrete. Due to the temporary nature of the proposed wall, a system of earth reinforcement that would be even more economical than those previously constructed was required.

A MSE wall similar to that constructed in Dunsmuir on Interstate 5, as reported by Chang, et al (1981), had potential for construction at the Delhi site. Experience had shown that soil reinforcement systems such as MSE develop an apparent soil pressure at the face which is somewhat less than the theoretical design pressure due to redistribution of stress. As a result of this experience, it was reasoned that wood-facing could provide an economic yet adequate alternative to either steel or concrete facing for this project.

The WFMSE built at Delhi was Caltrans' first experience utilizing a temporary wall of this type. However, a similar permanent treated wood-faced wall of lower height was constructed in 1978 at Santa Barbara on Route 192 and has performed satisfactorily. Due to the experimental nature of the Delhi wall, it was instrumented to monitor short-term performance. Details of the instrumentation appear in later sections of this report.

It should be noted that MSE was developed by the California Department of Transportation and is licensed under a joint agreement with the Reinforced Earth Company. In accordance with that agreement, an alternative design for a RE wall with steel facing was submitted for this project by the Reinforced Earth Company. However, the alternative selected for construction by the low bidder was the wood-faced MSE.

## B. Elements Comprising Wood-Faced Mechanically Stabilized Embankment (WFMSE)

WFMSE is essentially a bar-mat reinforced soil mass in which the strength and weight of the backfill, coupled with the reinforcement-soil frictional resistance and mobilized passive resistance of soil due to transverse reinforcement, provide internal stability for the embankment.

Important elements in design of MSE are the selection and sizing of the reinforcement and wall-facing. The system can be utilized with essentially any backfill material that is nonexpansive, i.e., plasticity index (PI) is less than 10 (Hannon and Forsyth, 1984).

The reinforcing system designed for the WFMSE at Delhi consisted of bar-mats using 11 individual W5 wires (0.252 inch diameter) placed longitudinally (perpendicular to the wall face) at 6-inch spacings with tack welded W5 transverse wires at 20-inch spacings (Figure 4). However, the contractor elected to use prefabricated W5 bar-mats with 6x12 grid spacings which were readily available. The bar-mats were fabricated in two lengths, 10 feet for upper portion of wall (higher than 12 feet) and 15 feet for lower portion of wall (refer to Figure 4). The 11 longitudinal wires were welded to a 1/4 inch x 2-3/4 inch x 5 foot-4 inch plate which, in turn, was welded to three 3/4-inch diameter threaded rods that provide connection to the wood-face. These connection assemblies were shop fabricated. The facing materials were 1-1/8 inch x 4 foot x 8 foot plywood panels positioned vertically with the long dimension in the horizontal direction. Where long mats (15 feet) were used, up to the wall height of 12 feet above the top of the footing, two rows of panels were utilized, back-to-back

(Figure 4). The panels were provided additional stability by 4 inch x 6 inch, undressed, Douglas fir, vertical posts attached to the front face of the plywood panels at two foot spacings (Figure 4). The height of the panels and the vertical posts varied with the embankment height.

The vertical posts were positioned and cast in a one foot square continuous, unreinforced concrete footing and were braced to maintain their vertical position.

The material for the entire embankment was imported from a local source. No special selection of backfill material was required for the wall. The material consisted mostly of silty sands and sandy silts with the following strength properties:

Table 1

Strength Properties of Backfill

| Unconsolidated Undrained Test (UU) | Consolidated Drained Test (CD) | Unit Weight ($\gamma$) |
|---|---|---|
| $\phi$ = 36 degrees | $\phi$ = 39.5 degrees | Wet = 125 pcf |
| c = 800 psf | c = 0 | Dry = 112 pcf |

An evaluation of the interaction of the soil and reinforcement was not required for this project because the strength properties of the soil were considered more than sufficient for internal stability based on pullout tests with other materials. Additional pullout tests were therefore not necessary. Internal stability could be achieved by providing embedment depth of reinforcement to satisfy external stability, i.e., overturning factor of safety of 2.0 and factor of safety against sliding of 1.5. For additional information on the interaction of soil and reinforcement for both MSE and RE refer to the report by Chang, et al (1977) which discusses pullout test results on steel bar-mat and steel strip reinforcement.

The wall design for this project assumed backfill material with angle of internal friction ($\phi$) of 32 degrees, cohesion (c) of zero and soil density ($\gamma$) of 130 pcf. A sloping backfill condition was also assumed above the wall with partial saturation of backfill. An active Rankine triangular pressure distribution with $K_a$=0.46 was utilized.

## C. Instrumentation

To monitor the performance of this wall, the following instruments were installed at Station 365$\pm$:

1. Strain gages on the bar-mats and anchorage bolts to check the stress history in the reinforcement and connections,

2. Gloetzel soil presure cells behind the wood facing to check the earth pressures within the embankment, and

3. Strain gages to check the stresses on the vertical posts and plywood facing.

The relative locations of the instrumented levels and the individual instruments are shown in Figure 5.

Due to the short-term life of this wall (8 months), many of the basic evaluations that are usually undertaken on experimental projects, i.e., corrosion loss, settlement and horizontal movement, etc., were not conducted.

D. Laboratory Installation of Instruments

1. Strain Gages on Bar-Mats

Four bar-mats were laboratory instrumented prior to installation at four different levels in the embankment. The location and identification of each strain gage is shown in Figure 6. The process of attaching the Ailtech 120 weldable strain gages to the bar-mats began by first preparing the designated gage location by sanding to remove all oxidation. The strain gages were then welded at a rate of 30 welds per inch (Photo 1). No protective covering for moisture or physical contact was applied to the gages. Each instrumented bar-mat was outfitted with a wood frame to prevent gage damage during transit to the project.

2. Strain Gages on Vertical Posts

One 4 inch x 6 inch x 24 foot vertical timber post member was laboratory instrumented with strain gages installed at the four different instrumentation levels of the wall. The location and identification of each individual strain gage is shown in Figure 7. The surface of the wood member at the gage locations was sanded before attaching the PML Polyester 120 type strain gages. A coating of EPY 150 epoxy was then applied to the wood surface and the strain gages were attached (Photo 2). All surfaces of the member then received a coating of epoxy to prevent moisture penetration.

### 3. Strain Gages on Plywood Panels

Six 4 foot x 8 foot plywood sheets, labeled A through F, were laboratory instrumented with the PML Polyester 120 type strain gages. The location and identification of each gage is shown in Figure 7. Each set of strain gages was centered horizontally and located three feet from the left edge of each of the instrumented panels. The procedure for attaching the gages to the plywood panels was the same as that for the posts (Photo 3). All instrumented panels were treated with epoxy paint to eliminate moisture change in the wood as another variable influencing panel response in the field.

## E. Determination of Elastic Properties of Steel Bar-Mats and Wood Members

### 1. Elastic Property of Bar-Mats

#### a. Material Specifications

Plans for the Delhi WFMSE called for use of W5 wire bar-mats with the configuration shown in Section B-B, Figure 4. As previously discussed, the contractor elected to use a 6 inch x 12 inch grid in lieu of 6 inch x 20 inch grid.

The wire-mesh reinforcement for the bar-mats complied with ASTM Designation A-185 for welded steel wire fabric for concrete reinforcment which specified the following for W5 wire:

Nominal diameter: 0.252 inch
Tensile strength, min (KSI) = 75
Yield strength, min (KSI) = 65

b. Testing Procedures and Results

A bar-mat sample was obtained and tested for strength properties and modulus of elasticity according to ASTM Designation A82. The test specimen provided a yield strength of 77.2 ksi and a tensile strength of 79.4 ksi which complied with specifications. Analysis of the tensile test data revealed a modulus of elasticity of $29.3 \times 10^6$ psi.

2. Elastic Properties of Douglas Fir Posts

The setup for load testing the wood posts is shown in Figure 8 and Photo 4. The post was instrumented with PML polyester 120 type strain gages as described above in Section D.2. Since the wood member's cross-section varied due to drying shrinkage, the relative spacings of the strain gages varied from group to group. Deflection pots were installed under each strain gage group to measure the displacement of the wood member at these points.

With all instrumentation in place, the post was loaded by a hydraulic jack. A load cell between the jack and the post indicated the applied load on an electronic readout. After calibration, the mechanism was advanced by hand jacking until the required load was displayed on the readout. Data from the strain gages and deflection pots were recorded, and the load was increased to the next level. Deflections were recorded at 25 lb load increments up to 200 lb. The modulus of elasticity was determined for four individual wood members using classical mechanics of materials analysis for a simply supported beam undergoing elastic deformation according to the equation shown in Figure 8. The load-deflection relationship for the instrumented wood post is shown in Figure 9.

The calculated moduli of elasticity for the wood posts are summarized in Table 2 and are compared with a standard value for timber construction. The instrumented post showed a close correlation, coming within 2% of the standard value. The average experimental modulus of elasticity from four tests was greater than the standard value by 7.8%. This variation may be explained by a change in cross-section as a result of shrinkage, moisture content variation and the nonhomogenous nature of construction timber.

The theoretical relationships for calculation of modulus of elasticity of the posts are shown in Appendix A.

Table 2

Summary of Moduli of Elasticity of Douglas Fir Posts

| Post Designation | Modulus of Elasticity, E, (psi) | E, From Timber Construction Manual, (psi) |
|---|---|---|
| Instrumented Post | $1.77 \times 10^6$ | $1.8 \times 10^6$ |
| A | $2.14 \times 10^6$ | $1.8 \times 10^6$ |
| B | $1.65 \times 10^6$ | $1.8 \times 10^6$ |
| C | $2.20 \times 10^6$ | $1.8 \times 10^6$ |

All posts were nominal 4x6 select structural grade Douglas fir.

### 3. Elastic Property of Plywood Panel

#### a. Material Specifications and Testing Procedure

Plywood is composed of veneer sheets of wood, compressed and glued together in layers that alternate in grain direction by 90°. Usually, there is an odd number of veneers so that the surface layers have a similar grain direction (see Figure A1 in Appendix B). Since the strength characteristics of wood are not isotropic but vary with grain direction, the moment of inertia (I) and modulus of elasticity (E) vary with the direction of the applied stress. In this experiment, the properties of the plywood panel were tested in the laboratory with the applied load producing stresses parallel to the surface grain of the panel.

As described previously, under test procedure for the wood posts, the modulus of elasticity of the plywood was determined by loading the member and measuring the deflections, then applying the relationship between load, deflection, and E. Gages were installed on the plywood panel to measure strains both perpendicular and parallel to the applied load. The plywood panel was simply supported at both ends and a line load applied in the middle of the span as shown in Figure 10. Also refer to Photo 3.

Three extensometers spaced evenly under the line loading recorded panel deflection for test loads applied in 25 lb increments up to 200 lb. Polyester gages glued to the plywood panel recorded strains at each load level for comparison with theoretical strains. Tests were run on six panels, labeled A through F, for which calculated values of moment of inertia, modulus of elasticity, and section modulus, are shown in Table 3. Typical load-deflection plots are shown in Figure 11.

TABLE 3

TEST DATA SUMMARY - PLYWOOD PANELS A THROUGH F

| Panel | Thickness of Plies, Inches $t_1$ | $t_2$ | $t_3$ | $t_4$ | $t_5$ | $t_6$ | $t_7$ | $P/\Delta$ | Moment of Inertia $I_{11}$, $in^4/ft$ | Modulus of Elasticity $E_{11}$, psi | Section Modulus KS, $in^3/ft$ | Percent Change $\frac{\sigma}{\varepsilon Cal.}$ to $\frac{\sigma}{\varepsilon Measured}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 0.123 | 0.168 | 0.144 | 0.208 | 0.155 | 0.200 | 0.136 | 443.7 | 0.8916 | $2.0957 \times 10^6$ | 1.738 | +0.6% |
| B | 0.116 | 0.137 | 0.173 | 0.182 | 0.145 | 0.174 | 0.125 | 406.8 | 0.7750 | $2.2109 \times 10^6$ | 1.2737 | +1.3% |
| C | 0.131 | 0.182 | 0.163 | 0.155 | 0.161 | 0.188 | 0.134 | 370.6 | 0.8912 | $1.7514 \times 10^6$ | 1.3615 | +44.5% |
| D | 0.133 | 0.186 | 0.156 | 0.249 | 0.158 | 0.144 | 0.131 | 425.6 | 1.0165 | $1.7634 \times 10^6$ | 1.4687 | +30.4% |
| E | 0.125 | 0.168 | 0.163 | 0.200 | 0.163 | 0.170 | 0.113 | 390.0 | 0.8447 | $1.944 \times 10^6$ | 1.3174 | +42.2% |
| F | 0.118 | 0.198 | 0.159 | 0.193 | 0.149 | 0.189 | 0.124 | 351.5 | 0.8775 | $1.687 \times 10^6$ | 1.3115 | +17.8% |
| Average | 0.124 | 0.172 | 0.161 | 0.198 | 0.149 | 0.178 | 0.127 | 398.0 | 0.8828 | $1.909 \times 10^6$ | 1.412 | +22.8% |

$E_{11}$ (parallel to grain) for all plies = $1.8 \times 10^6$ psi

| | Tabulated Plywood Design Specification Value | Experimentally Calculated Average |
|---|---|---|
| $E_{11}$ | $1.8 \times 10^6$ psi | $1.909 \times 10^6$ psi |
| $I_{11}$ | 0.676 $in^4/ft$ | 0.883 $in^4/ft$ |
| KS | 1.047 $in^3/ft$ | 1.412 $in^3/ft$ |

b. Data Correlation

Due to the empirical nature of the values listed in the Timber Construction Manual (TCM), our experimental values seem best suited for this analysis. They indicate that the tabulated values for design are on the conservative side and probably take into account the worst allowable case for plywood of a given type. A comparison of calculated experimental values and specified design values for plywood construction are shown at the bottom of Table 3.

The average modulus of elasticity of the plywood of $1.9 \times 10^6$ psi found experimentally for this project is within 6 percent of the value given in the TCM. The average values for moment of inertia and section modulus are within 35 percent of the TCM values. The tabulated values for moment of inertia and section modulus given in the TCM are "effective" values and "...have been adjusted to account for several variables..." (APA, 1978, p. 12).

As expected, the gages placed parallel to the load read zero strain for all load increments indicating that no torsion was occurring in the panels.

The measured (from instrumentation) and calculated moduli of elasticity are shown in Table 4. Panels A and B show very close agreement between measured and calculated values and Panels C, D, E and F show a reasonable correlation. Refer to Figure 11 for a plot of the load-deflection relationships for Panels A, B and C. For all panels tested (Table 3), the stress versus measured strain had a greater slope than the stress versus calculated strain. This indicates that the measured strain was smaller than the calculated strain for all loading increments. This would seem to indicate that panels C through F were actually more elastic than expected and that the actual value for the modulus of elasticity was greater than the calculated value.

Table 4

Measured and Calculated Moduli of Elasticity of Plywood Panels

| Panel | Measured E, psi | Calculated E, psi |
|---|---|---|
| A | $2.08x10^6$ | $2.10x10^6$ |
| B | $2.24x10^6$ | $2.21x10^6$ |
| C | $2.10x10^6$ | $1.75x10^6$ |
| D | $2.00x10^6$ | $1.76x10^6$ |
| E | $2.30x10^6$ | $1.94x10^6$ |
| F | $1.98x10^6$ | $1.69x10^6$ |

## F. Vacuum Loading Tests on Plywood Panels

### 1. Testing Procedures and Results

A vacuum load test was devised based on information from Dallas and Mitzner (1977). This type of test was felt to be a good, inexpensive method for evaluating the performance of the plywood panels and strain gages under the type of loading expected in the actual field installation. It was felt that the test results could be compared to pressure cell measurements in the field. An advantage of this type of test is that uniform loads can be applied safely and measured accurately.

Two of the instrumented plywood panels (Panels A and B), were subjected to vacuum tests for determination of their modulus of elasticity and reaction to pressure loading. This was accomplished by utilization of a special test frame (Photos 5 and 6). The two panels were separated at their perimeter by a welded aluminum channel frame, forming a thin rectangular box. Within this box configuration, wooden posts were placed, as illustrated in Figure 12, to simulate actual construction conditions of the wood-faced wall. A sealant dispensed from a caulking gun was applied to the contact surfaces between the plywood and the aluminum frame. Both panels were also sealed on their outside surfaces with epoxy paint. Holes were bored and bolts, washers, and nuts were installed at the actual locations required for bar-mat fastening in the field.

Vacuum was applied to the box through fittings installed in the aluminum frame. A maximum vacuum of 18 inches (8.84 psi) of Hg (mercury) was applied and strain measurements were obtained at various increments between 0 and 18 inches of Hg. These data are summarized in Table 5 and are plotted in terms of actual strains on Figure 13.

2. Correlation Between Theoretical and Actual Strains

The reader is referred to Appendix C for information on the theoretical development of strains.

In the horizontal direction, the theoretical and the actual laboratory measured strains were -696 μin/in and -725 μin/in, respectively, for Panel A and -906 μin/in and -991 μin/in, respectively, for Panel B. These values were determined at a pressure of 18 inches of Hg and correspond reasonably well.

TABLE 5

RESULTS OF LABORATORY VACUUM TESTS ON PLYWOOD PANELS A & B

Strain vs Vacuum Pressure

| Vacuum Pressure | | Strain, ε (μ in./in.) | | | |
|---|---|---|---|---|---|
| | | Panel "A" | | Panel "B" | |
| | | Channel | | Channel | |
| Inches of Hg | (psi) | 4 (Horiz.) | 5 (Vert.) | 6 (Horiz.) | 7 (Vert.) |
| 0 | 0 | 0 | 0 | 0 | 0 |
| 4.25 | 2.09 | -155 | 17 | -190 | 1 |
| 6 | 2.95 | -230 | 28 | -299 | -2 |
| 8 | 3.93 | -324 | 33 | -427 | -6 |
| 10 | 4.91 | -417 | 42 | -559 | -10 |
| 12 | 5.89 | -493 | 51 | -663 | -15 |
| 14 | 6.87 | -591 | 59 | -804 | -22 |
| 16 | 7.86 | -666 | 67 | -908 | -26 |
| 18 | 8.84 | -725 | 75 | -991 | -28 |

In the vertical direction, the theoretical and actual strain values did not correspond nearly as well. In Panel A, the theoretical and actual strains were -20 μin/in and 75 μin/in, respectively, and in Panel B they were -19 μin/in and -28 μin/in, respectively. Correlation was almost nonexistent. This behavior could again be attributed to the irregular and nonhomogeneous wood plies.

## G. Calibration of Strain Gages

### 1. Strain Gages on Bar-Mats

The linearity of the strain gages installed on the bar-mats was tested in the laboratory under tensile loads.

A nut with a hook-fastening feature was threaded on the center bolt connector of each instrumented bar-mat prior to testing. Each mat was lifted individually with an overhead hoist until it was vertical (Photo 7). The single wire strand with the strain gages attached was threaded at the opposite end and fastened to a chain of testing paraphernalia that included a load cell, hydraulic jack, and anchors. The jack applied the tension load and the load cell monitored the load application.

Once a mat was in position as described above, testing began by first applying a load of 1,000 lb (the maximum to eventually be applied), and then releasing the load. Load applications in increments of 100 lb were then applied from 0 to 1,000 lb with strain readings obtained at each 100 lb increment. This testing procedure was repeated three times with each instrumented bar-mat.

### 2. Strain Gages on Wood Posts

The laboratory test setup and procedure was described previously in Section E.2. and the theoretical relationships are presented in Appendix D. Also refer to Photo 4.

The difference between measured strain and calculated strain was evaluated for all strain gage loadings for calibration purposes. This difference was expressed as a percentage of the calculated strain. The average percentage deviation for each loading was used as a parameter for comparing the accuracy of each gage.

Better correlation was found for gages closer to the top surface of the member during loading. A few large variations between measured and theoretical strain gage values probably suggest that wood members are not perfectly homogeneous and exhibit some inelastic behavior. It is interesting to note that in most cases the measured strain and theoretical strain for a given gage were almost equivalent at the point of applied load.

## H. Calibration of Soil Pressure Cells

The Gloetzel soil pressure cells were calibrated in the laboratory prior to incorporation in the project. Each cell was individually placed in the large air pressure calibration chamber at the Transportation Laboratory and tested at chamber pressures from zero to 30 psi. In addition, three pressure cells were placed in direct sunlight and surface temperature of the cells was allowed to raise to 140°F. Maximum change in prepressure readings of the cells from the indoor ambient temperature condition with cell surface temperature of 90°F to outdoor direct sun conditions was 0.01 percent.

## I. Wall Construction

### 1. General

Wall construction began on March 2, 1979, and was completed in 18 working days. During two of the working days, high wind conditions slowed work progress. After the first few days, a routine was established that provided optimum work progress for the number of workers and equipment involved. This routine consisted of dividing the 1300 foot wall length in half to form two work areas. As the crew of laborers and carpenters fastened panels, drilled holes and placed bar-mats in one work area, embankment placement proceeded in the other. This system accelerated progress and contributed greatly to the rapid completion of the wall.

Prior to wall erection, the planned bar-mat configuration was altered by a Contract Change Order. The spacing of the transverse bars was changed from 20 inches to 12 inches center to center. The contractor requested this change because the latter mat configuration was readily available.

### 2. Placement of Concrete Footing and Erection of Wood Facing Members

Excavation for the footing was accomplished with a trenching machine that excavated a 1 foot x 1 foot trench (Photo 8). The excavated trench walls contained the concrete during placement and no forms were required. Prior to placement of the footing concrete, the 4 inch x 6 inch vertical wood posts were positioned and nailed to the plywood panels at 2 foot centers in units of four posts to one 4 foot x 8 foot sheet of plywood (Photo 9). The plywood sheets (panel) were positioned such that when the wood units were erected vertically, the prenailed panels became the second row of face members from the bottom, thereby allowing room for footing placement. These wood units were then erected and placed in the proper position in the trench with the aid of a truck-mounted telescoping boom with cable slings. Laborers guided the wood units into position and nailed them to the adjoining wood units (Photo 10). This newly positioned wood unit was then braced with lengths of Douglas fir struts nailed to wooden stakes driven in the ground (Photos 11 and 12). The trenching operation progress ahead of the vertical whaler erection operation was controlled so that at the end of the work day there was no open trench area. Concrete placed in the trench formed a nominal 1 foot x 1 foot footing with the lower 1 foot of each 4x6 post embedded in concrete (Photo 13).

Installation of bottom level plywood panels began as soon as the footing concrete had cured a minimum of one day (Photo 14). Once the bottom panels were in place, the first level of holes were drilled in the panels to accommodate the connection bolts for the bar-mats (Photo 15). Embankment was placed and compacted to a level just below the row of bolt holes and the bar-mats were then distributed on the newly placed embankment behind the wall with bolt equipped ends inserted into the appropriate holes (Photo 16). Washers and nuts were fastened to the bolts and tightened with a pneumatic wrench from the outside of the wall to snug the bar-mats flush with the back face of the panels (Photo 17). As the nut tightening activity progressed to higher levels, the process was conducted from the back of a flatbed truck and finally from scaffolding housed on the truck bed. The truck also served to pull the trailer-mounted compressor that supplied the air for the pneumatic wrench.

This cycle of plywood erection, bar-mat placement, embankment placement was continued alternately in both work areas until the full wall height was attained. Plywood erection was kept just far enough ahead of embankment placement to permit the carpenters and laborers to work conveniently while standing on the new embankment.

3. Backfill Placement and Compaction

Embankment (backfill) material, delivered to the job site by bottom dumps, was spread and then compacted by a vibratory roller (Photo 18). A water truck complemented the compaction operation. The truck and trailer traffic also contributed to the compactive effort. A road grader aided in material spreading and leveling to grade. A hand-operated vibratory steel roller provided compaction within two feet of the wall face (Photo 19). The degree of compaction was monitored by nuclear gage.

4. Field installation of Instrumented Members

a. Instrumented Wood Post

Following the laboratory attachment and calibration of strain gages to the wood post, it was wrapped in fiberglass fabric insulation to protect the gages and wires and transported to the job site. The instrumented post was erected at its proper location as part of a wood unit consisting of four posts and one sheet of plywood (refer to Photo 20). The protective insulation material on the instrumented post was unwrapped only long enough to accommodate panel attachment. Strain readings were obtained before embankment was placed against the post, and during and after wall construction.

b. Instrumented Plywood Panels

The six instrumented panels were installed by Transportation Laboratory personnel and located as illustrated in Figure 7. Each panel was installed just prior to embankment placement. Precautions were taken to prevent damage to strain gages during this operation.

c. Instrumented Bar-Mats

Each of the four instrumented bar-mats was installed by Transportation Laboratory personnel at the proper level after the embankment reached the appropriate elevation (refer to Figure 5). The embankment material consisted primarily of sand and silt with no appreciable amount of coarse material. Typical grading values included 100% passing No. 4 and about 35% passing the No. 200 sieve. This type of material was very desirable for bedding and protection of the instrumented mats. With little effort, the material was screened to produce a fine-graded bedding material for the exposed strain gages. This finer graded material was placed over the gages as a cushion from any coarser material and construction equipment loadings.

d. Soil Pressure Cells

Ten Gloetzel soil pressure cells were installed on the embankment side of face panels as shown in Figure 5. The manufacturer recommended placing the cells against the panels so that they would be in the vertical plane of the panel face and no cells would project out from the face. It was also necessary to provide a uniform, smooth contact between cell and panel.

Thus, in the laboratory, housings for the cells were carved out of the panels in the specific locations where they were to be installed, and in the field, the actual installation took place (Photo 21). The cells were kept in place prior to embankment placement by small tabs.

5. Construction Costs

The simplicity of the wood-faced wall erection and the efficient operation of embankment placement resulted in a short construction period and contributed to a substantial overall construction cost reduction. Except for the initial vertical post erection (where a truck-mounted telescoping boom and a trenching machine were used), special equipment was not required during wall construction and embankment placement. Wood face panels were nailed in position before embankment was placed against them and steel bar-mats were manually placed. The silty sand embankment material allowed easy placement and reduced compaction time considerbly. Construction costs are tabulated in Table 6 and show an actual unit construction cost of $6.18/ft$^2$ for the wood-faced wall. Photo 22 shows a view of the wall nearing completion.

## J. Analysis of Instrumentation Data

1. Soil Pressure Distribution

No established theory was available at the time of design to precisely analyze soil pressures in WFMSE. The design for the Delhi wall utilized the closest applicable and yet conservative theories of retaining wall design as described in Section 5.b.

## TABLE 6

## CONSTRUCTION COSTS

| | |
|---|---|
| CLEARING AND GRUBBING | $ 794 |
| Includes: equipment, survey and labor | |
| MECHANCIALLY STABILIZED EARTH WALL | $ 75,800 |
| Includes: equipment, steel bar-mats, plywood panels, Douglas fir posts, concrete footing, survey and labor | |
| TRENCH EXCAVATION | $ 1,940 |
| Includes: equipment, survey and labor | |
| EMBANKMENT | $ 60,500 |
| Includes: equipment, backfill material, survey and labor | |
| TOTAL COST | $139,034 |
| FACE AREA | 22,500 $ft^2$ |
| ACTUAL CONSTRUCTION COST PER SQ FT OF FACE AREA* | $6.18/$ft^2$ |
| TOTAL ESTIMATED SAVINGS WHEN COMPARED TO OTHER ALTERNATIVES | about $300,000 |

*The Caltrans estimate was $8.65/$ft^2$

At-rest pressure theory is not considered applicable to MSE design because of the flexible nature of the system. It is, therefore, appropriate to design the system based on the active earth pressure and assume a certain amount of pressure redistribution behind the wall as defined by Chang, et al (1981).

The initial design of this wall assumed a conservative active Rankine state of stress with a horizontal active pressure coefficient, $K_a$, of 0.46 ($\phi$=32 degrees, c=0 and $\gamma_{wet}$=130 pcf) for a sloping backfill condition determined from the following equation:

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

where: $\beta$ = angle of backfill slope above wall

$\phi$ = angle of internal friction of backfill

The above $K_a$ value (0.46) represents the completed wall with embankment and roadway above it. The May 2, 1979 condition is representative of wall completion to full height with no sloping backfill or surcharge. The analysis which follows is based on the May 2, 1979 condition.

Nuclear soil densities measured in the field during wall construction suggest $\gamma_w$=120 lbs/ft$^3$. Strength parameters with angle of internal friction $\phi$ (36 degrees) and c (0) were also assumed for the actual backfill condition. The Rankine active coefficient, $K_a$, at wall completion is determined as follows for the zero surcharge condition:

$$K_a = \frac{1-\sin 36°}{1+\sin 36°} = 0.26$$

The lateral soil pressure at 20 foot wall depth (H), is calculated as

$$p = \frac{K_a \gamma H}{144} = \frac{0.26(120)(20)}{144} = 4.3 \text{ psi}$$

Ten soil pressure cells were installed on the inside face of the wall to measure horizontal (lateral) pressures at five different elevations for comparing actual field results to the above theory (Photo 21). Initial measurements indicated higher than normal horizontal pressures that were in excess of theoretical values. These higher pressures were due, in part, to the construction operation. However, within a couple of days of installation, the pressure cells indicated a decrease in horizontal pressure. All cells except two returned to zero pressure. Retrieved sections of plywood panels, containing 8 of the 10 pressure cells, obtained several months after construction revealed soil penetration into the contact between the cells and the panels in almost all cases. This suggests that the panels may have distorted or warped slightly and fines from the backfill soil migrated into the voids between the cells and the wood facing which may have resulted in somewhat erroneous readings.

Figure 14 shows a plot of measured pressure readings versus overburden height during construction as well as the theoretical active pressure discussed earlier. Higher pressures were recorded in the top portion of the wall, perhaps due to the built-in cantilever effect above the common splice location in the vertical posts (refer to Figure 14, CG-5, CG-10 and Photo 23). The contractor failed to tighten the bolts for these splice connections until after the first lift of soil and bar-mat reinforcement were placed above the splice. The top of the wall tilted outward approximately 12 inches as a result of embankment placement and equipment loading prior to tightening the bolt connections. The vertical alignment could not be maintained above the splice connections during construction. No measurable additional outward movement occurred following construction.

No attempt has been made to assess the cantilever effect on the lateral pressure, but it is assumed that actual pressures would have been somewhat less with the vertical alignment. Higher internal pressures were also recorded during the early stages of loading at each level, possibly due to excessive compaction near the face. However, these lateral pressures dissipated as wall construction progressed. The lateral pressure near the toe (CG-1, Level 1) remained constant during wall construction possibly due to toe restraint. Alternative procedures for predicting soil pressure were used that applied data recorded from strain gages installed on panels, posts and barmats. They are discussed in the sections which follow.

### 2. Stresses in Plywood Panels

As mentioned previously, the WFMSE included instrumentation to measure strain in the plywood panels. These strains were measured by polyester strain gages (as described in Section D) at various wall heights and on both sides of the wall face. The location of the gages is shown in Figure 7.

Strain gage readings were used to compute plywood panel stresses, define the soil pressure applied to the plywood panels, and compare the results with both the theoretical and measured soil pressures.

As shown in Figures 15 and 16, the strains in the plywood panels increased significantly after completion of the wall on May 2, 1979 and continued to increase through the final readings taken in October 1979. The highest strain readings were measured on the inside face of the wall (Figures 15b and 16b). These excess strain readings did not occur in the vertical posts (Section J.3.) or in the bar-mats and thus cannot be attributed entirely to loading. The strain gages on the plywood may have experienced creep, partial penetration of moisture, or both. Also, the bar-mat connection assemblies installed flush with the back of face could have induced panel warpage.

The data shown in Table 7 were developed from the May 2, 1979, strain readings which appear reasonable.

Both calculated and estimated soil pressures are presented in Table 7 using recorded strain readings from the horizontal gages. The calculated lateral soil pressures in Column 5 were determined from field strain readings assuming the face panels were acting as continuously loaded beams with supports at two-foot intervals (refer to Appendix C). Column 6 in Table 7 shows values of lateral soil pressure estimated from laboratory vacuum tests (Figure 13) and adjusted for double panel units. A good correlation exists between the calculated and estimated values of soil pressure.

TABLE 7

Stresses and Strains in Plywood Face Panels

| Panel & Level** | Gage | Location* | May 1, 1979 Strain, in./in.x$10^{-6}$ | Calculated Soil Pressure, psi | Estimated Soil Pressure Based on Vacuum Test, psi |
|---|---|---|---|---|---|
| A-1 | 21 | H-O | 48 | 2.81 | 2.4 |
| | 22 | V-O | -103 | -- | -- |
| B-1 | 23 | H-I | -14 | 0.63 | 0.8 |
| | 24 | V-I | 465 | -- | -- |
| C-2 | 25 | H-O | 35 | 1.34 | 1.2 |
| | 26 | V-O | -26 | -- | -- |
| D-2 | 27 | H-I | 100 | 4.16 | 4.0 |
| | 28 | V-I | -168 | -- | -- |
| E-3 | 29 | H-O | 118 | 4.86 | 4.8 |
| | 30 | V-O | -79 | -- | -- |
| | 31 | H-I | -69 | 2.84 | 2.4 |
| | 32 | V-I | +60 | -- | -- |
| F-4 | 33 | H-O | 220 | 2.38 | 2.2 |
| | 34 | V-O | -19 | -- | -- |
| | 35 | H-I | -307 | 3.33 | 3.1 |
| | 36 | V-I | -187 | -- | -- |

*H-O (Horizontal gage - Outside of wall)
V-O (Vertical gage - Outside of wall)
H-I (Horizontal gage - Inside of wall)
V-I (Vertical gage - Inside of wall)

**Levels 1, 2 and 3 have double panels, Level 4 has single panel

Vertical strain gage readings were not utilized in this analysis since they appear to be unreliable as an indicator of lateral soil pressure (refer to Section F.2.)

Figure 17 illustrates the range in lateral soil pressure of calculated and estimated values for different overburden heights. A comparison is also presented for the triangular Rankine, $K_a$, pressure distribution. Even though no correlation existed between the two distributions of pressure, the maximum pressures (theoretical vs calculated) are in close agreement.

Additional analysis of soil pressure is covered in subsequent sections of this report.

### 3. Stresses in the Wood Posts

As part of the wall facing, the Delhi MSE included 4 inch x 6 inch Douglas fir posts up to 24 feet long and spaced at 2-foot intervals. The pattern of bar-mats attached with bolt connectors alternated between members. At 4-foot vertical intervals, each post was anchored with a bar-mat connection bolt (Figure 4). At each interval between bar-mat members, the posts were bolted to the plywood panels. As shown in Photo 23, each wood post exceeding the 12-foot height was subject to a common splice at the 12-foot level.

The instrumented Douglas fir post was installed during wall construction at Station 365+00 and extended to the full height of wall without splicing. Five strain gages were located at each of four levels, corresponding to the four levels of instrumented bar-mats (refer to Figures 5 and 7). Strain histories for each of the strain gages are shown in Figure 18.

Contrary to results for the plywood panels, strains in the wood posts did not continue to increase after wall completion. The strain gage data in Figure 18 suggest that the instrumented post was subject to outward bending for its entire height. However, its performance is not representative of the other post members which had a common splice at the 12-foot height. The data are, therefore, provided only for information and no further analysis is attempted.

4. Analysis of Bolt Data

Data collected from strain gages installed on the bar-mat bolt connections (Gages 101 and 102, Level 1; 111 and 112, Level 2; 121 and 122, Level 3; and 131 and 132, Level 4) are very questionable since all transfer connection members were bolted flush against the back of the wood facing during construction (refer to Figure 6 and Section I.2. of this report). Thus, no attempt was made to analyze these data because of the preload condition and the data are presented in Figure 19 for information only.

5. Stresses in Bar-Mats and Correlation With Lateral Soil Pressures

Daily histories of stresses in the bar-mats calculated from measured strains are plotted in Figure 20. Stresses generally peaked at wall completion on May 2, 1979, and decreased following wall construction. The stresses then increased during placement of the structural section and later with traffic on the fill.

All bar-mat stresses were less than the maximum allowable working stress of 24 ksi. The maximum recorded stress was 19.5 ksi in Levels 2 and 3 near the center of the bar-mat at both levels in October 1979 prior to encapsulation of the wall in the embankment.

Figure 21 presents bar-mat stresses recorded at completion of the wall on May 2, 1979.

For estimating lateral soil pressures on the wall face, an equivalent pressure was determined from strain gages located one foot back of the wall face on May 2, 1979. The connector bolt stresses could not be utilized since they were not a good indicator due to the construction procedure.

Since an average of two bar-mats were placed per 4x8-foot face panel, each bar-mat supported a face area equal to 16 $ft^2$. Using the bar-mat strains measured on May 2, 1979, the pressure distribution shown in Figure 22 will result.

The calculated pressures appear to follow the Rankine triangular distribution in the lower portion of the wall. However, at Level 4, above the vertical spliced members, the pressure is somewhat higher, possibly due to the cantilever effect above the splice. The pressure distribution could also be rectangular (similar to that found in braced cuts in dry or moist sand Terzaghi and Peck, 1968). This suggests that additional study is necessary.

Figure 23 provides additional analysis by which to estimate lateral soil pressure with overburden height. A comparison is presented for calculated and estimated soil pressures using the information discussed in previous sections of this report. The pressure cell readings from Figure 14, average soil pressure determined from face panel strains at inside and outside faces (Figure 17), and soil pressures determined from bar-mat stresses (Figure 22) are shown for comparison. The theoretical Rankine triangular pressure distribution at $K_a$ equal to 0.26 and a rectangular (uniform) pressure distribution for braced excavations in sand [$P=0.65\ Ht\tan^2(45°-\frac{\phi}{2})$] are also provided for comparison.

It should be pointed out that the pressure cell data under low overburden heights ($<6$ foot) appear to provide overregistration and may not be meaningful.

An excellent correlation exists between calculated and estimated soil pressures. These data points also agree reasonably well with soil pressures derived from bar-mat stresses at Level 1 (18 foot overburden) and Level 3 (10 foot overburden).

Final analysis of Figure 23 suggests that all four procedures for predicting soil pressure conclusively show that the pressure diagram (distribution) follows a rectangular rather than a triangular shape for this particular project. However, this does not suggest that this project was underdesigned using the triangular distribution. The steel bar-mats and connection bolts are generally selected based on the maximum stress at the bottom of the wall.

The rectangular pressure distribution could provide some economy by requiring less steel for tensile reinforcement on future designs. A rectangular pressure distribution should probably be utilized for wood-faced MSE walls using fine grained cohesionless backfill.

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INDEX OF SHEETS



F-P099(44)

STATE OF CALIFORNIA

BUSINESS AND TRANSPORTATION AGENCY

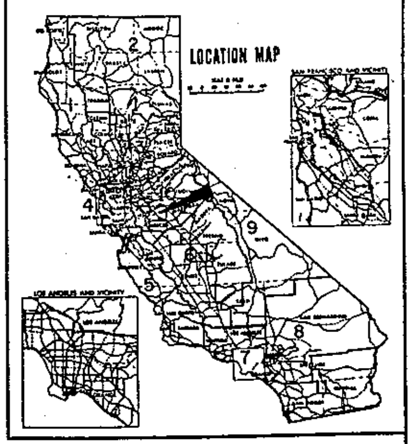
DEPARTMENT OF TRANSPORTATION

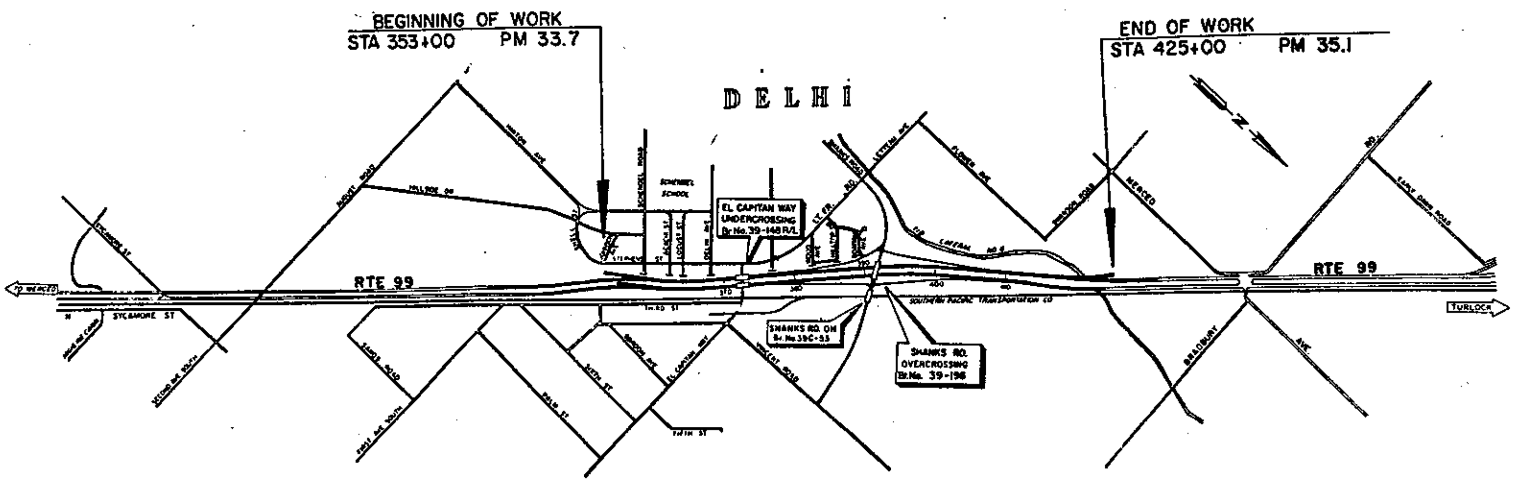
# PROJECT PLANS FOR CONSTRUCTION ON STATE HIGHWAY

IN MERCED COUNTY
AT AND NEAR DELHI FROM 0.4 MILE SOUTH OF
EL CAPITAN WAY TO 0.2 MILE NORTH OF TURLOCK
IRRIGATION DISTRICT LATERAL NO. 6

To be supplemented by Standard Plans dated March, 1977

| DIST | COUNTY | ROUTE | POST MILE | TOTAL PROJECT | |
|---|---|---|---|---|---|
| 10 | MER | 99 | 33.7-35.1 | 1 | 139 |




REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

SCALE IN FEET
0 500 1000 2000

D. L. Wieman
Registered Civil Engineer
Approved February 6, 1978

Length of Major Construction = 1.36 miles

Contract No. 10-043714

10203-04378

Figure 1

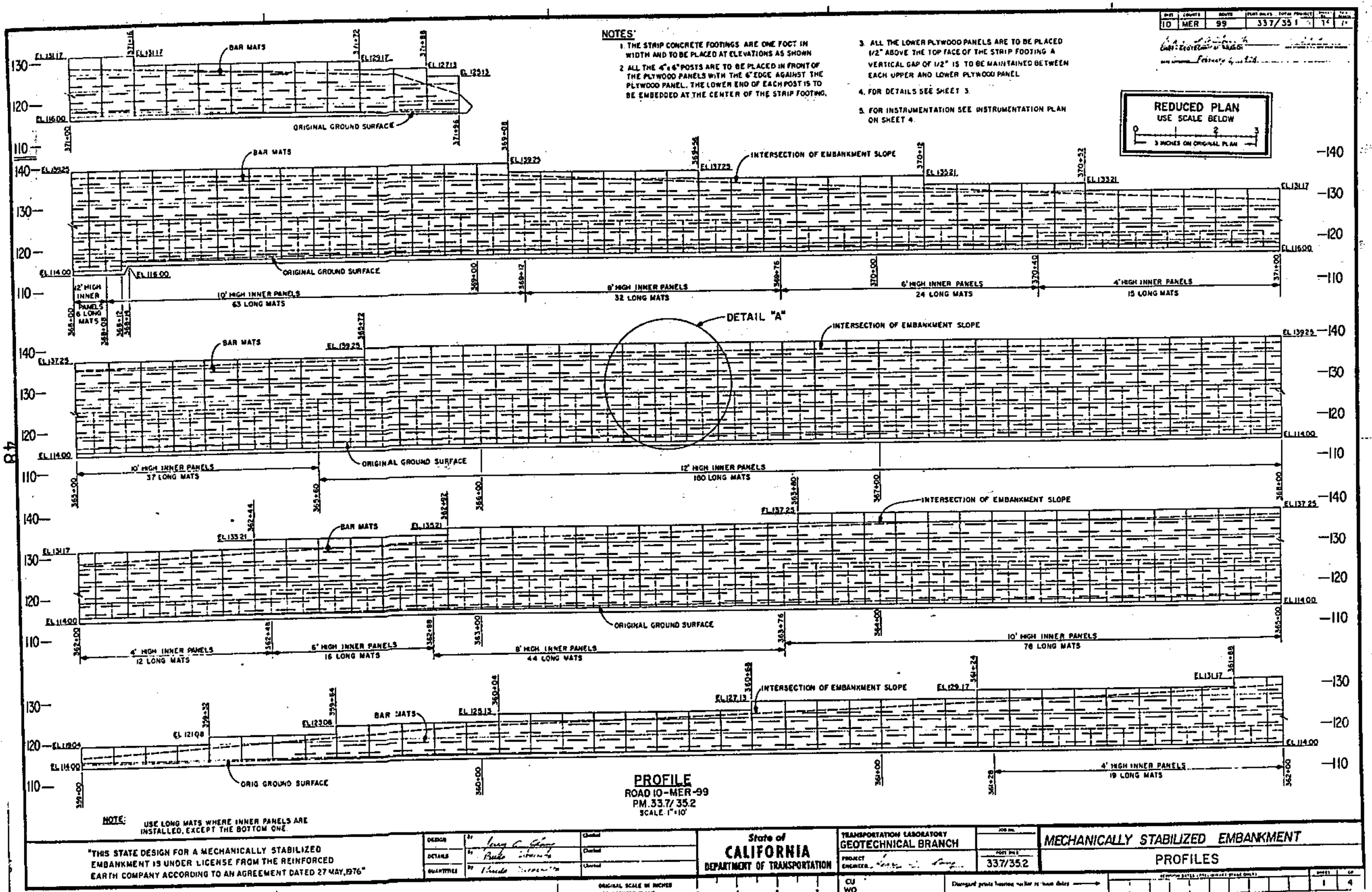
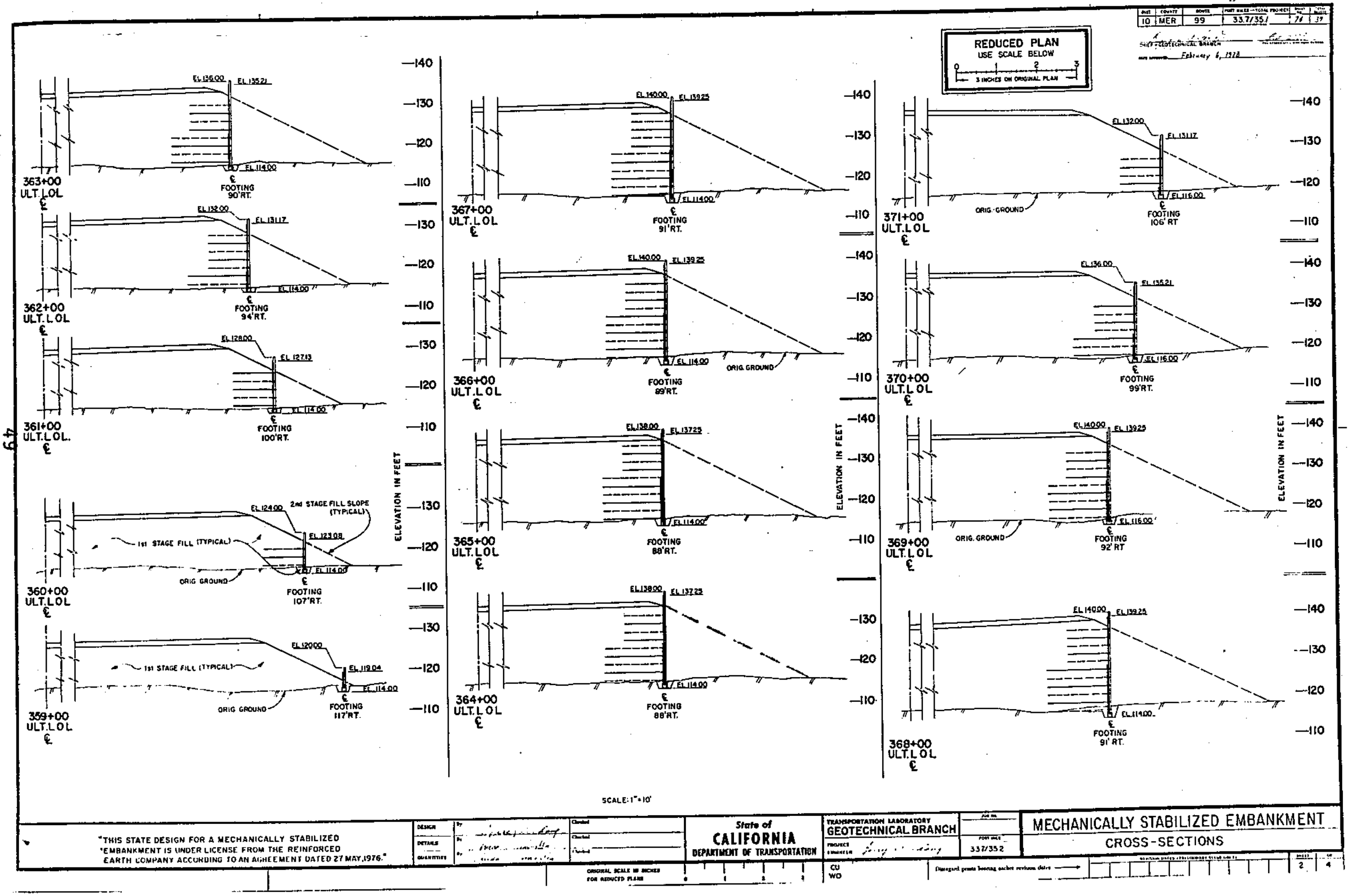

NOTES:
1. THE STRIP CONCRETE FOOTINGS ARE ONE FOOT IN WIDTH AND TO BE PLACED AT ELEVATIONS AS SHOWN
2. ALL THE 4"x6" POSTS ARE TO BE PLACED IN FRONT OF THE PLYWOOD PANELS WITH THE 6" EDGE AGAINST THE PLYWOOD PANEL. THE LOWER END OF EACH POST IS TO BE EMBEDDED AT THE CENTER OF THE STRIP FOOTING.
3. ALL THE LOWER PLYWOOD PANELS ARE TO BE PLACED 1/2" ABOVE THE TOP FACE OF THE STRIP FOOTING. A VERTICAL GAP OF 1/2" IS TO BE MAINTAINED BETWEEN EACH UPPER AND LOWER PLYWOOD PANEL
4. FOR DETAILS SEE SHEET 3.
5. FOR INSTRUMENTATION SEE INSTRUMENTATION PLAN ON SHEET 4.
REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN
BAR MATS
ORIGINAL GROUND SURFACE
INTERSECTION OF EMBANKMENT SLOPE
DETAIL "A"
12' HIGH INNER PANELS 6 LONG MATS
10' HIGH INNER PANELS 63 LONG MATS
8' HIGH INNER PANELS 32 LONG MATS
6' HIGH INNER PANELS 24 LONG MATS
4' HIGH INNER PANELS 15 LONG MATS
10' HIGH INNER PANELS 37 LONG MATS
12' HIGH INNER PANELS 180 LONG MATS
4' HIGH INNER PANELS 12 LONG MATS
6' HIGH INNER PANELS 16 LONG MATS
8' HIGH INNER PANELS 44 LONG MATS
10' HIGH INNER PANELS 78 LONG MATS
4' HIGH INNER PANELS 19 LONG MATS
ORIG GROUND SURFACE
PROFILE
ROAD 10-MER-99
PM. 33.7/35.2
SCALE 1"=10'
NOTE: USE LONG MATS WHERE INNER PANELS ARE INSTALLED, EXCEPT THE BOTTOM ONE.
"THIS STATE DESIGN FOR A MECHANICALLY STABILIZED EMBANKMENT IS UNDER LICENSE FROM THE REINFORCED EARTH COMPANY ACCORDING TO AN AGREEMENT DATED 27 MAY, 1976"
State of
CALIFORNIA
DEPARTMENT OF TRANSPORTATION
TRANSPORTATION LABORATORY
GEOTECHNICAL BRANCH
337/35.2
MECHANICALLY STABILIZED EMBANKMENT
PROFILES
10 MER 99 337/35.1

Figure 2



REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN
10 MER 99 337/351 74 39
February 6, 1978
363+00 ULT.LOL
FOOTING 90'RT.
EL 136.00
EL 135.21
EL 114.00
362+00 ULT.LOL
FOOTING 94'RT.
EL 132.00
EL 131.17
361+00 ULT.LOL.
FOOTING 100'RT.
EL 128.00
EL 127.13
360+00 ULT.LOL
FOOTING 107'RT.
EL 124.00
EL 123.08
2nd STAGE FILL SLOPE (TYPICAL)
1st STAGE FILL (TYPICAL)
ORIG GROUND
359+00 ULT.LOL
FOOTING 117'RT.
EL 120.00
EL 119.04
367+00 ULT.LOL
FOOTING 91'RT.
EL 140.00
EL 139.25
366+00 ULT.LOL
FOOTING 89'RT.
365+00 ULT.LOL
FOOTING 88'RT.
EL 138.00
EL 137.25
364+00 ULT.LOL
371+00 ULT.LOL
FOOTING 106'RT
EL 116.00
ORIG. GROUND
370+00 ULT.LOL
FOOTING 99'RT.
369+00 ULT.LOL
FOOTING 92'RT
368+00 ULT.LOL
FOOTING 91'RT.
ELEVATION IN FEET
SCALE: 1"=10'
"THIS STATE DESIGN FOR A MECHANICALLY STABILIZED
EMBANKMENT IS UNDER LICENSE FROM THE REINFORCED
EARTH COMPANY ACCORDING TO AN AGREEMENT DATED 27 MAY, 1976."
State of
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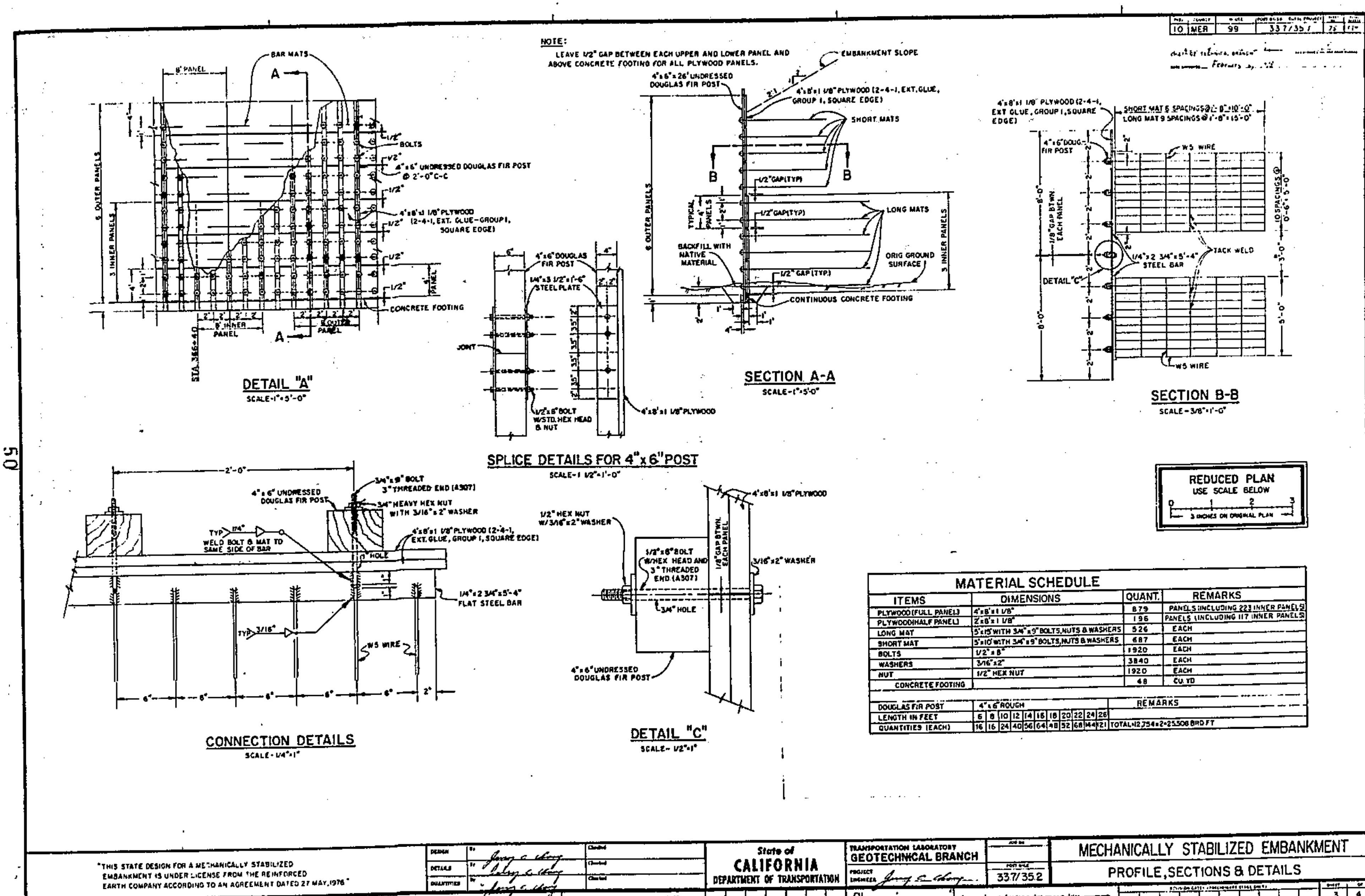
TRANSPORTATION LABORATORY
GEOTECHNICAL BRANCH
337/352
MECHANICALLY STABILIZED EMBANKMENT
CROSS-SECTIONS
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS


Figure 3




MATERIAL SCHEDULE

| ITEMS | DIMENSIONS | QUANT. | REMARKS |
|---|---|---|---|
| PLYWOOD (FULL PANEL) | 4'x8'x1 1/8" | 879 | PANELS (INCLUDING 223 INNER PANELS) |
| PLYWOOD (HALF PANEL) | 2'x8'x1 1/8" | 196 | PANELS (INCLUDING 117 INNER PANELS) |
| LONG MAT | 5'x15' WITH 3/4"x9" BOLTS, NUTS & WASHERS | 526 | EACH |
| SHORT MAT | 5'x10' WITH 3/4"x9" BOLTS, NUTS & WASHERS | 687 | EACH |
| BOLTS | 1/2"x8" | 1920 | EACH |
| WASHERS | 3/16"x2" | 3840 | EACH |
| NUT | 1/2" HEX NUT | 1920 | EACH |
| CONCRETE FOOTING | | 48 | CU. YD |

| DOUGLAS FIR POST | 4"x6" ROUGH | | | | | | | | | | | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LENGTH IN FEET | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | |
| QUANTITIES (EACH) | 16 | 16 | 24 | 40 | 56 | 64 | 48 | 52 | 68 | 144 | 21 | TOTAL=12,754x2=25,508 BRD FT |

"THIS STATE DESIGN FOR A MECHANICALLY STABILIZED EMBANKMENT IS UNDER LICENSE FROM THE REINFORCED EARTH COMPANY ACCORDING TO AN AGREEMENT DATED 27 MAY, 1976"

State of CALIFORNIA DEPARTMENT OF TRANSPORTATION

TRANSPORTATION LABORATORY GEOTECHNICAL BRANCH

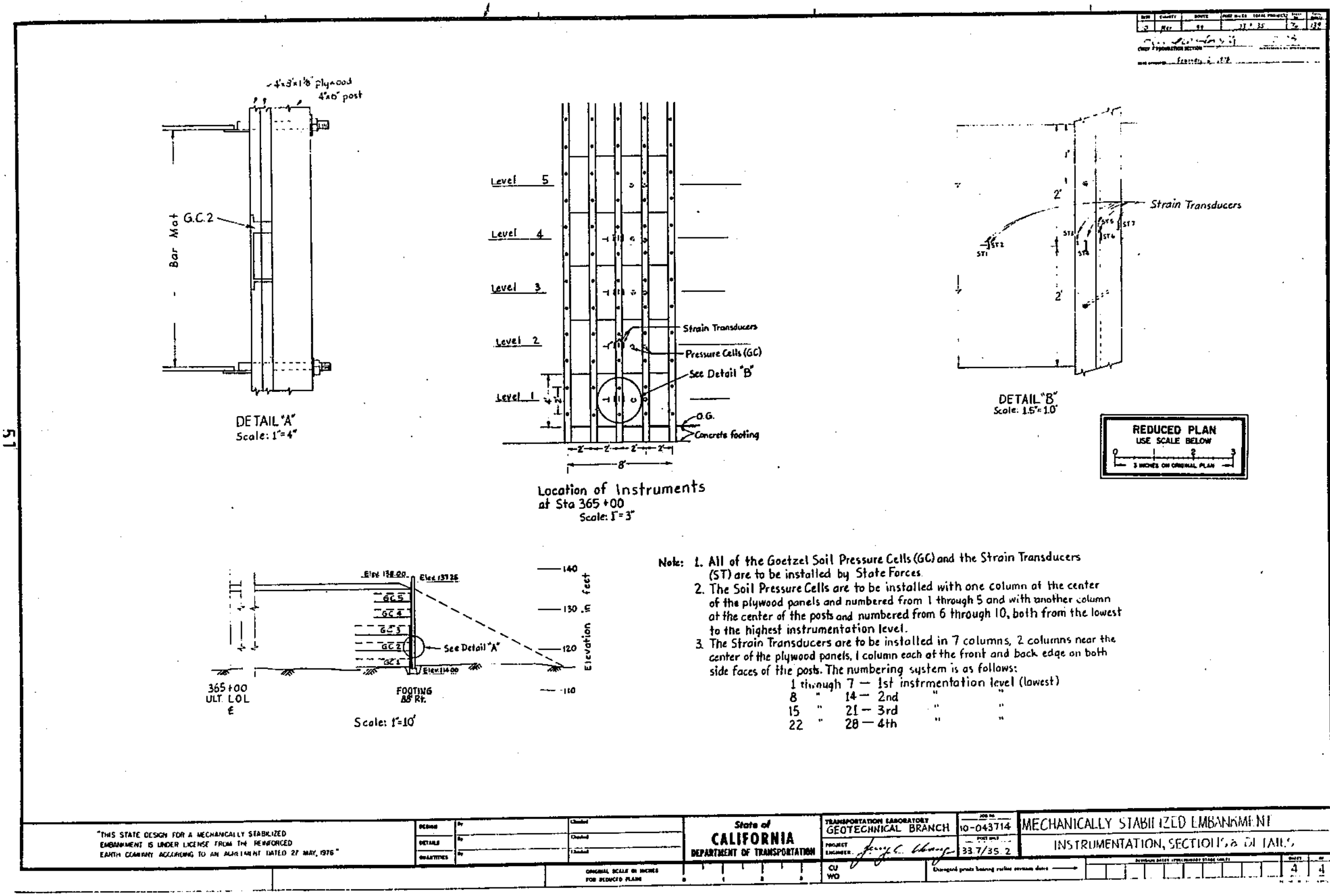
337/35.2

MECHANICALLY STABILIZED EMBANKMENT

PROFILE, SECTIONS & DETAILS

10 MER 99 337/35.1

Figure 4



4'x3'x1/8" plywood
4"x6" post
Bar Mat
G.C.2
DETAIL "A"
Scale: 1"=4'
Level 5
Level 4
Level 3
Level 2
Level 1
Strain Transducers
Pressure Cells (GC)
See Detail "B"
O.G.
Concrete footing
2' 2' 2' 2'
8'
Location of Instruments
at Sta 365+00
Scale: 1"=3'
Strain Transducers
2'
2'
DETAIL "B"
Scale: 1.5"=1.0'
REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN
Elev 138.00
Elev 137.36
GC5
GC4
GC3
GC2
GC1
See Detail "A"
Elev 114.00
365+00
ULT. LOL
℄
FOOTING
85' Rt.
Scale: 1"=10'
140
130
120
110
Elevation in feet
Note: 1. All of the Goetzel Soil Pressure Cells (GC) and the Strain Transducers (ST) are to be installed by State Forces.
2. The Soil Pressure Cells are to be installed with one column at the center of the plywood panels and numbered from 1 through 5 and with another column at the center of the posts and numbered from 6 through 10, both from the lowest to the highest instrumentation level.
3. The Strain Transducers are to be installed in 7 columns, 2 columns near the center of the plywood panels, 1 column each at the front and back edge on both side faces of the posts. The numbering system is as follows:
1 through 7 — 1st instrmentation level (lowest)
8 " 14 — 2nd " "
15 " 21 — 3rd " "
22 " 28 — 4th " "
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TRANSPORTATION LABORATORY
GEOTECHNICAL BRANCH
10-043714
33.7/35.2
MECHANICALLY STABILIZED EMBANKMENT
INSTRUMENTATION, SECTIONS & DETAILS
4 4


Figure 5

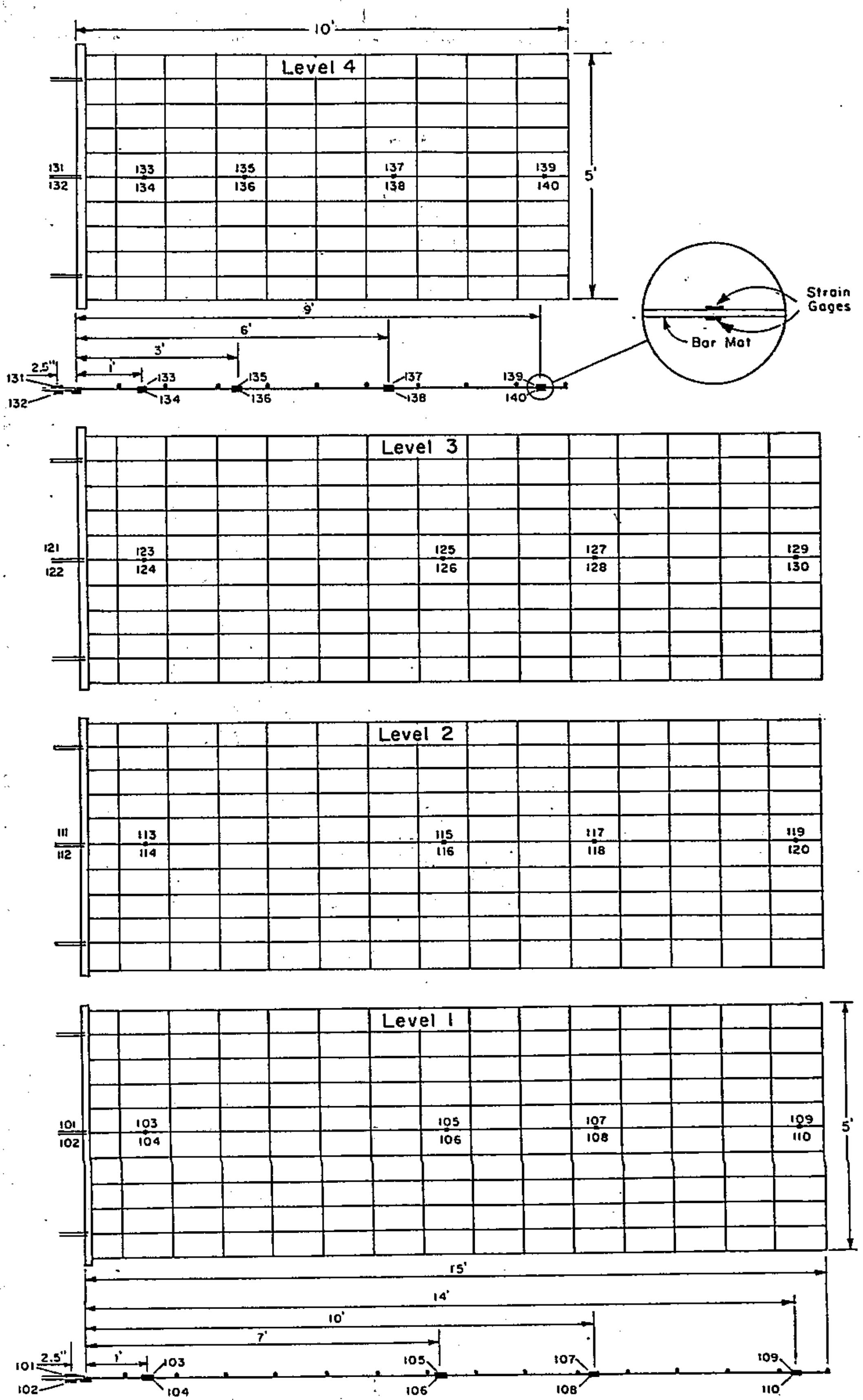
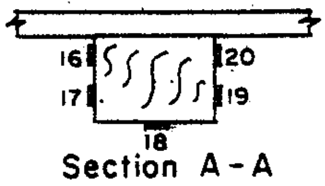



Fig. 6. STRAIN GAGE LOCATIONS ON BAR-MATS

Notes:

- ⊥ Strain Gages - horizontal and ertical
- ■ Soil pressure cells on fill side

(1) Panels D & B are inside panels with gages on the fill side
(2) Gages on panels A & C are on the outside
(3) Panels E & F have gages on both sides
(4) Gages 31, 32, 35, 36, are on the fill side
(5) Strain gages on post are located as shown by cross section



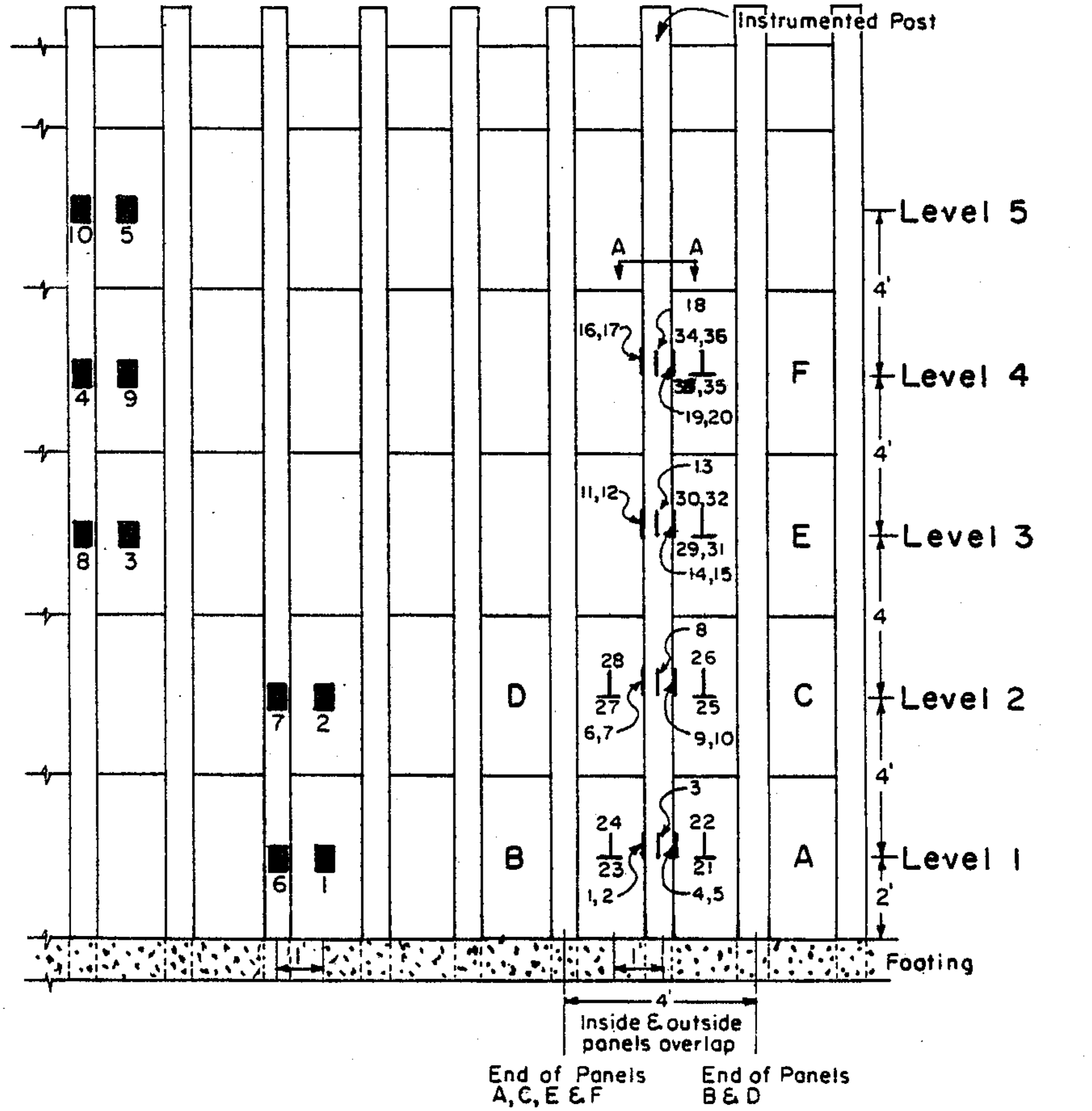
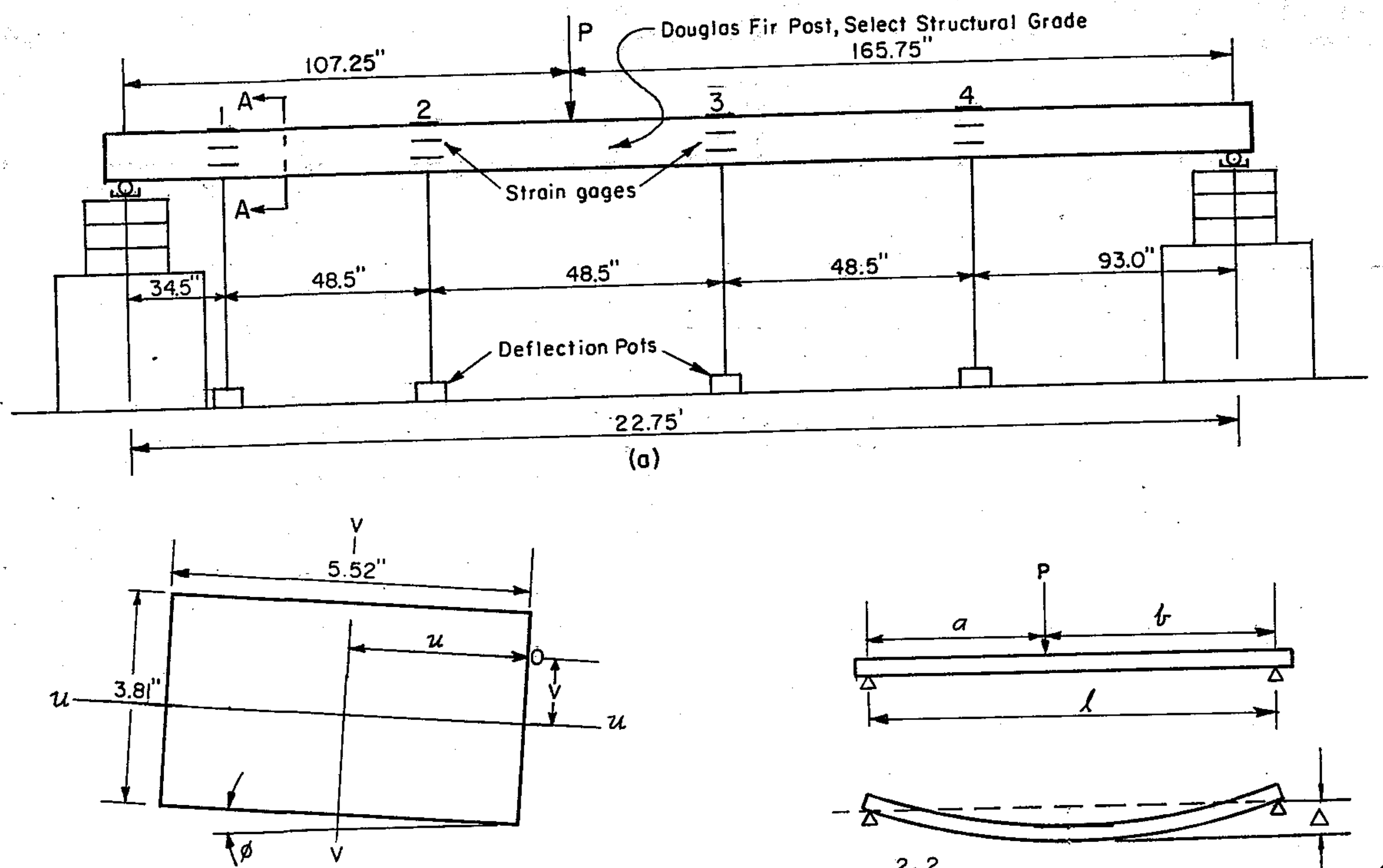




Fig. 7. POSITION OF STRAIN GAGES ON WOOD FACING MEMBERS & SOIL PRESSURE CELLS BEHIND FACING (Sta 365 ±)




Section A A Typical Cross Section
(b)

$\Delta = \frac{Pa^2b^2}{3EI\ell}$ where I = Moment of Inertia, in.$^4$

Load-Deflection Condition
(c)

Fig. 8. TEST ARRANGEMENT & THEORY FOR ELASTIC PROPERTIES OF TIMBER POSTS

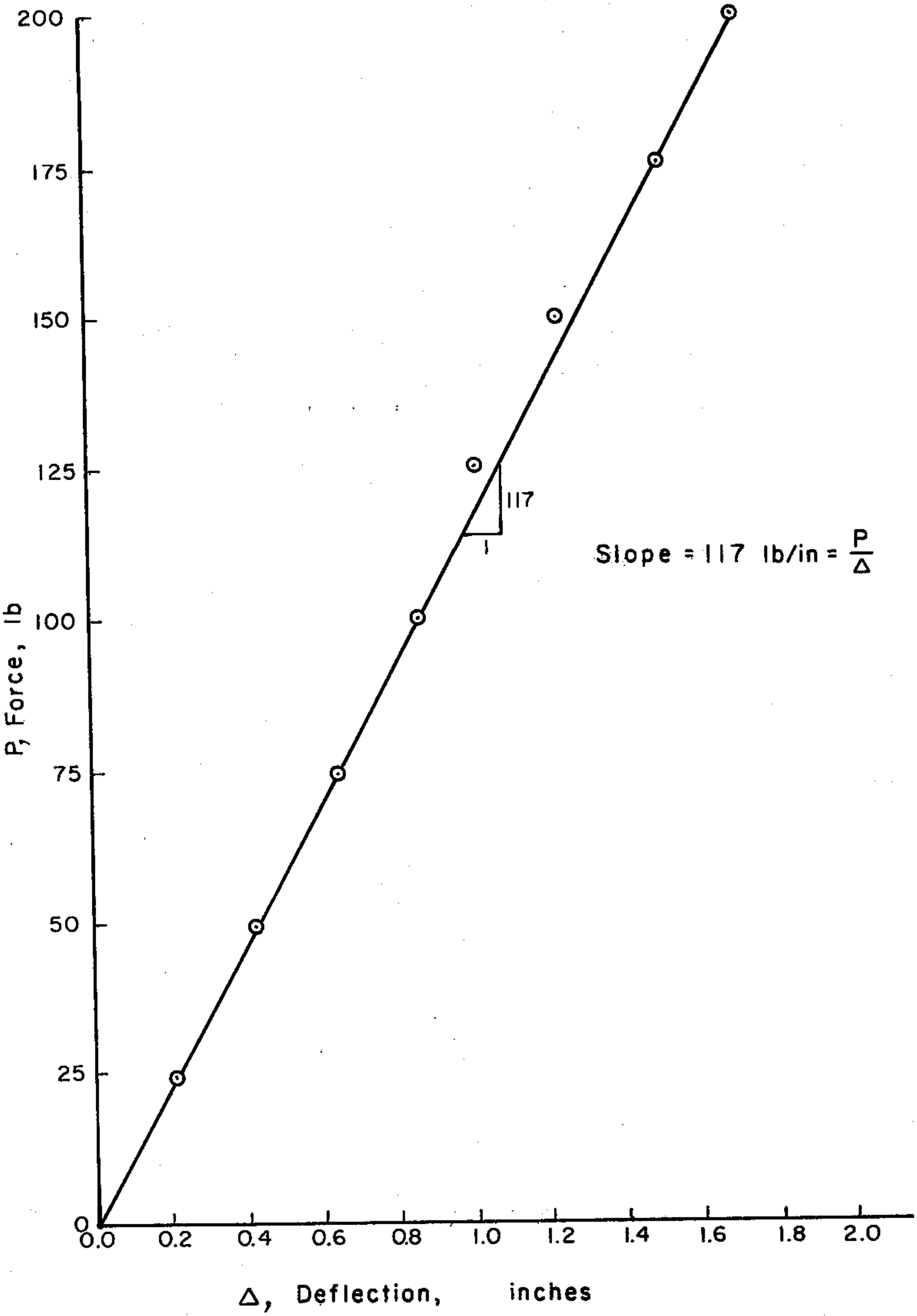



Figure 9. LOAD DEFLECTION RELATIONSHIP FOR INSTRUMENTED 4 in. x 6 in. DOUGLAS FIR POST

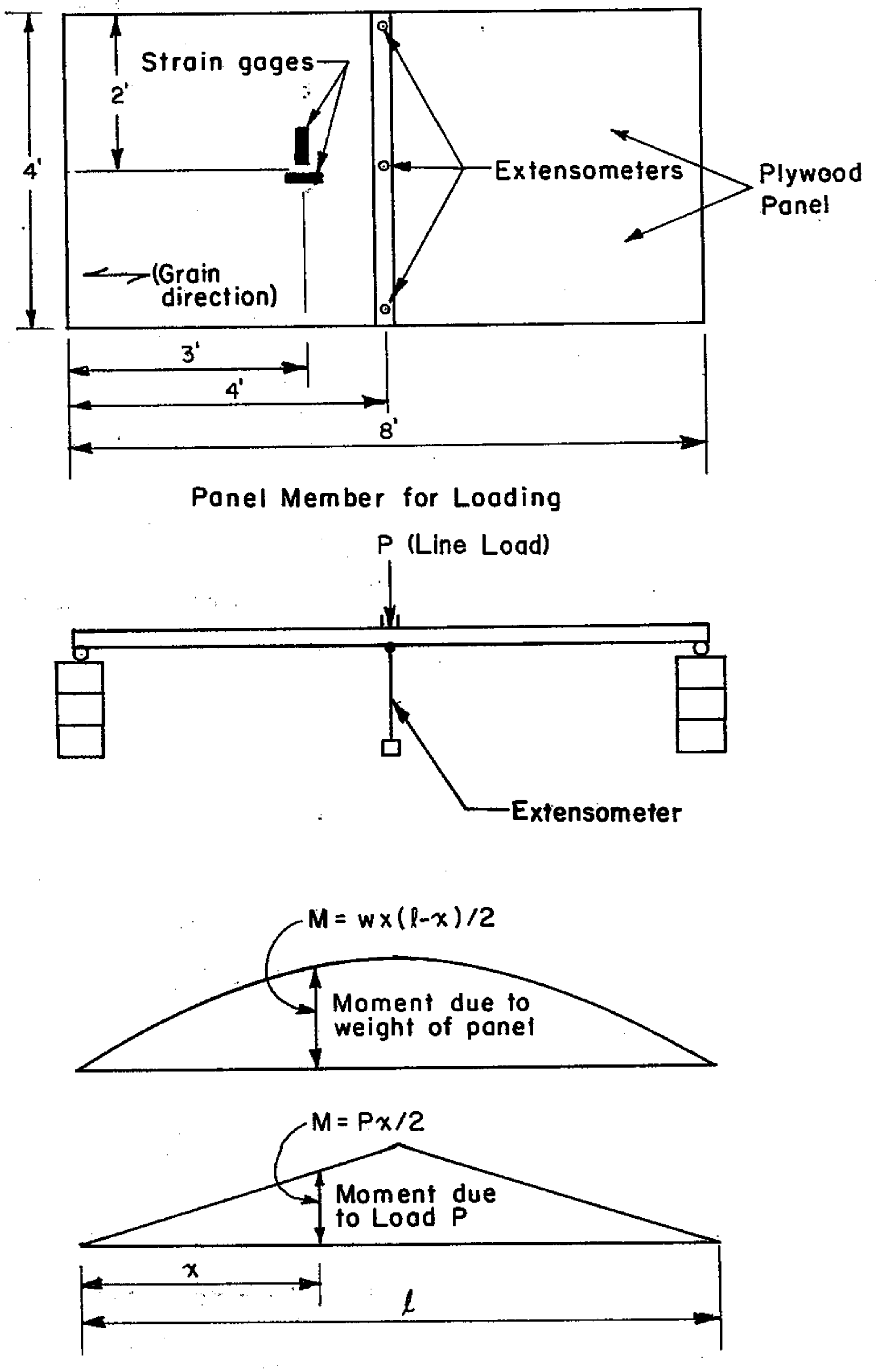



Fig.10. LOAD TEST & THEORY FOR ELASTIC PROPERTIES OF PLYWOOD PANELS

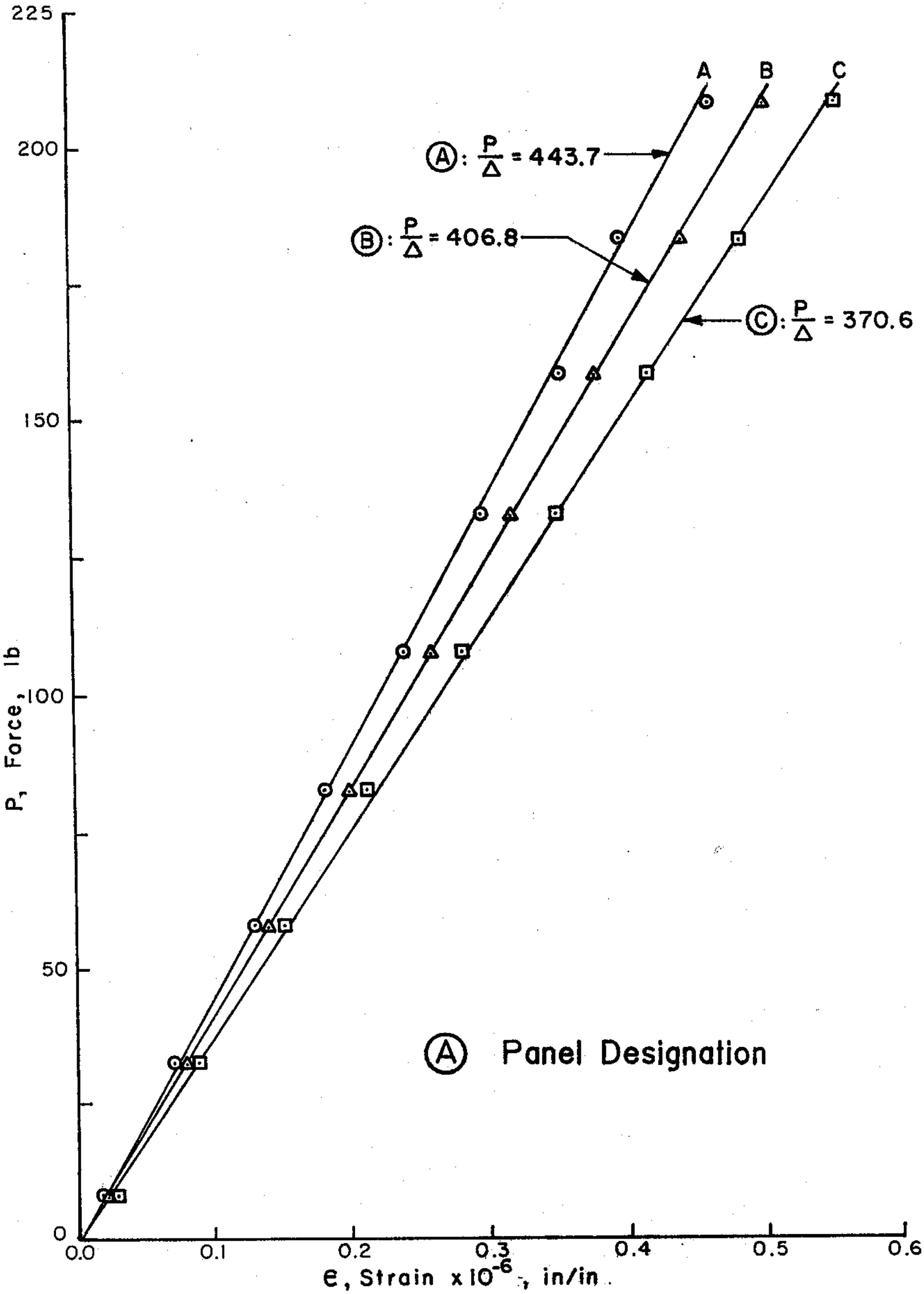



Figure 11. TYPICAL LOAD-DEFLECTION RELATIONSHIP, PLYWOOD PANELS A, B & C

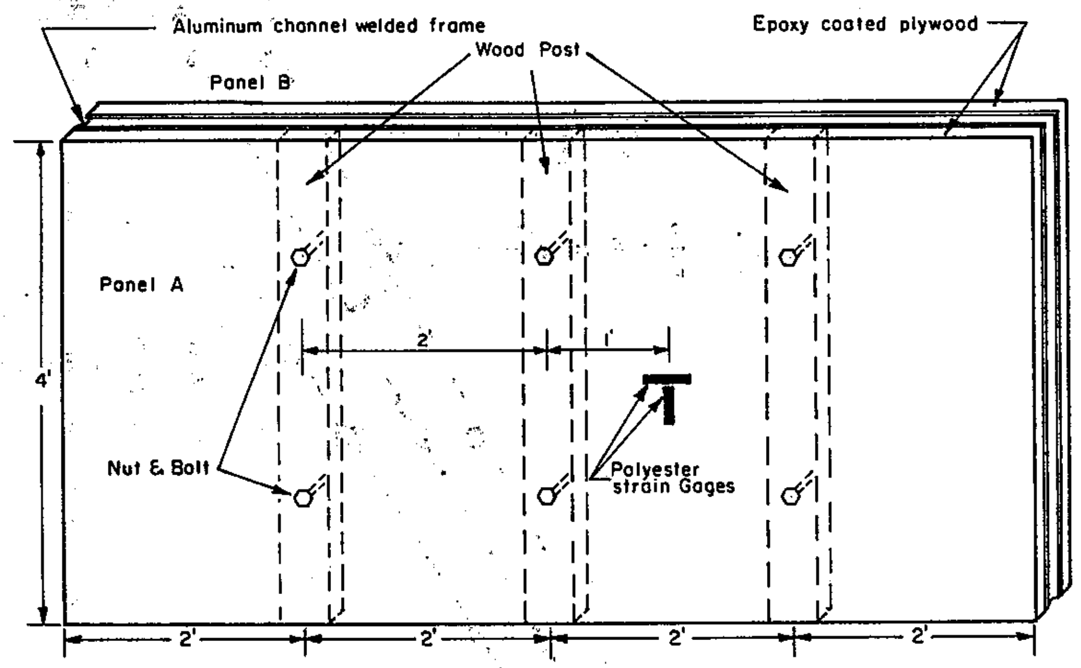



Fig. 12. VACUUM LOAD TEST FRAME FOR PLYWOOD FACE PANELS

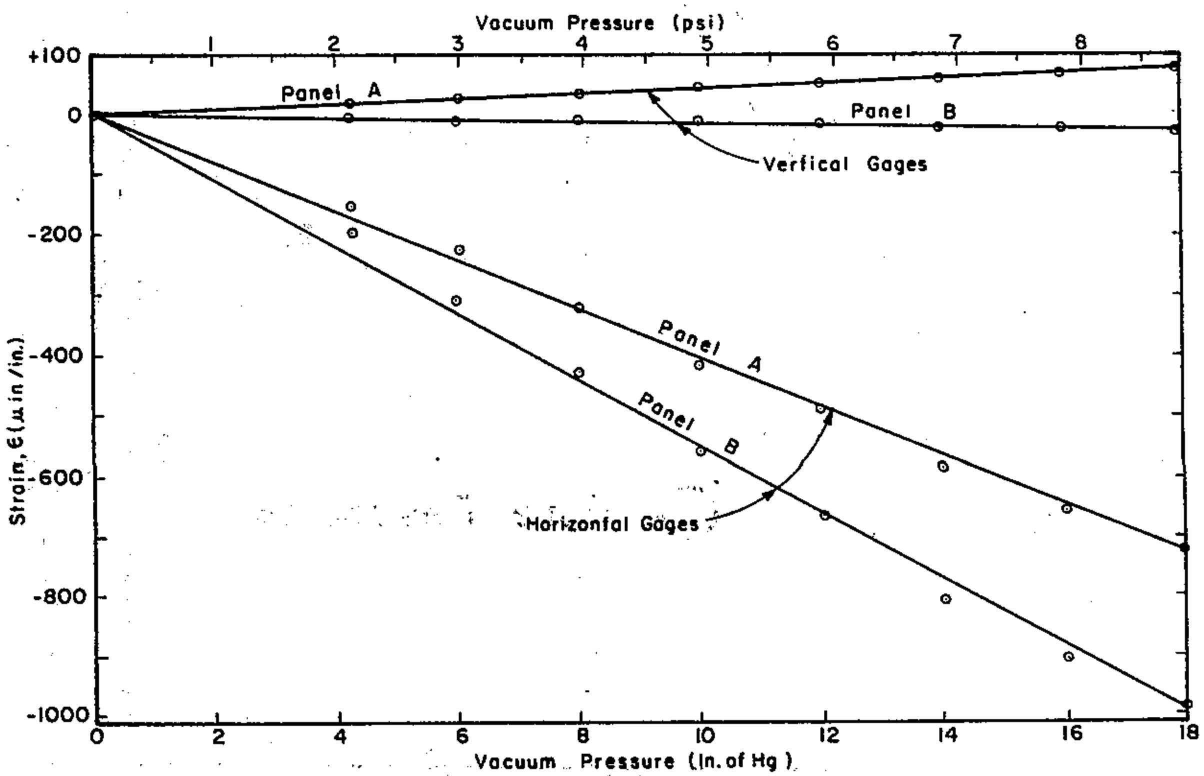



Fig. 13. STRAIN vs VACUUM PRESSURE FOR PLYWOOD PANELS A & B

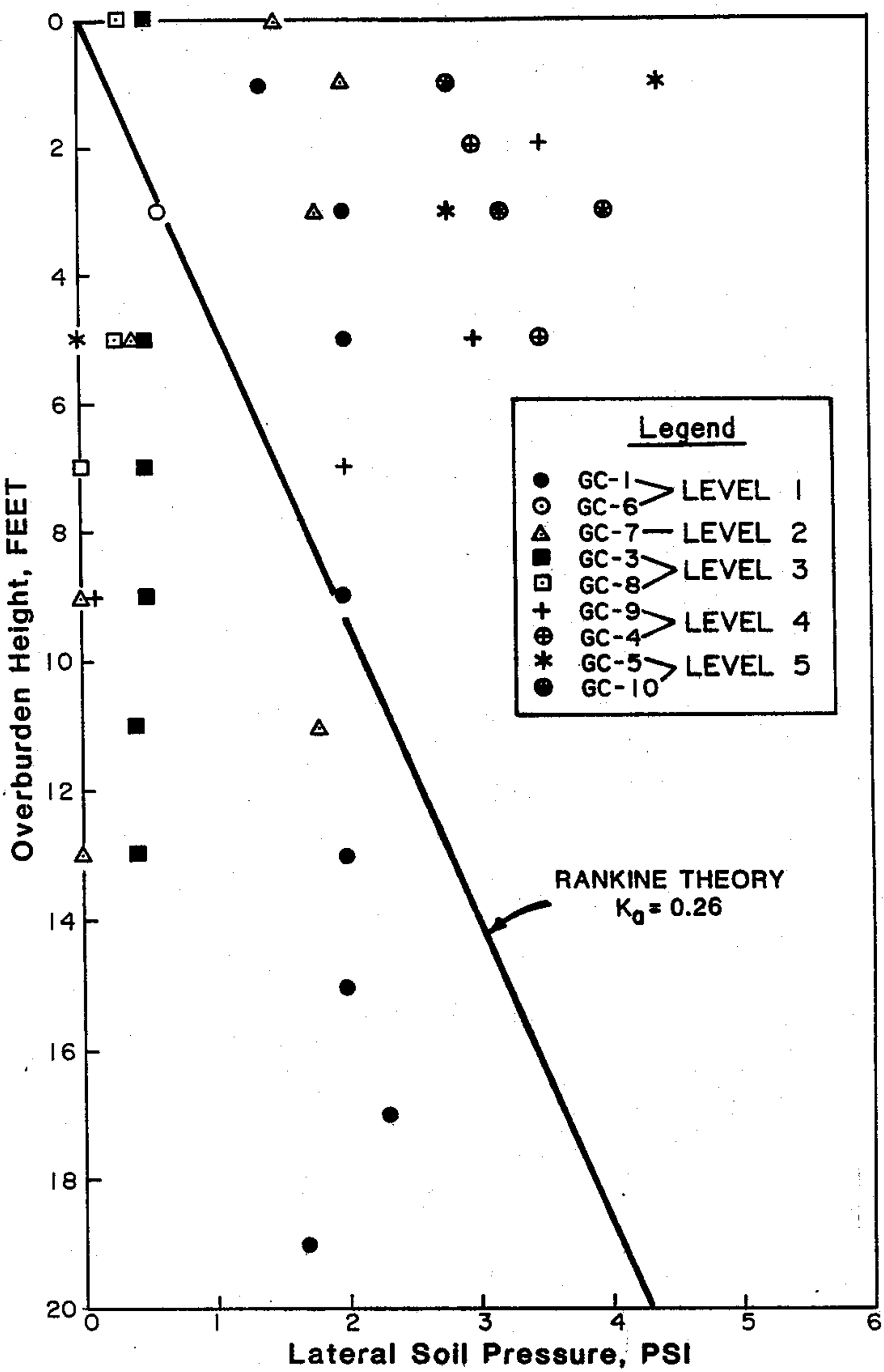
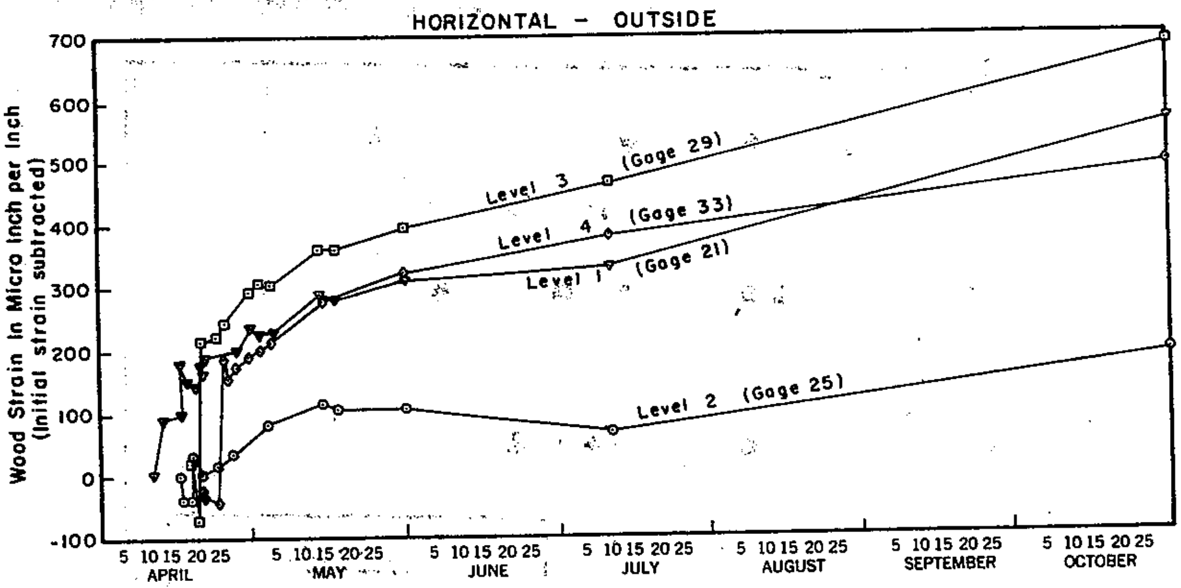
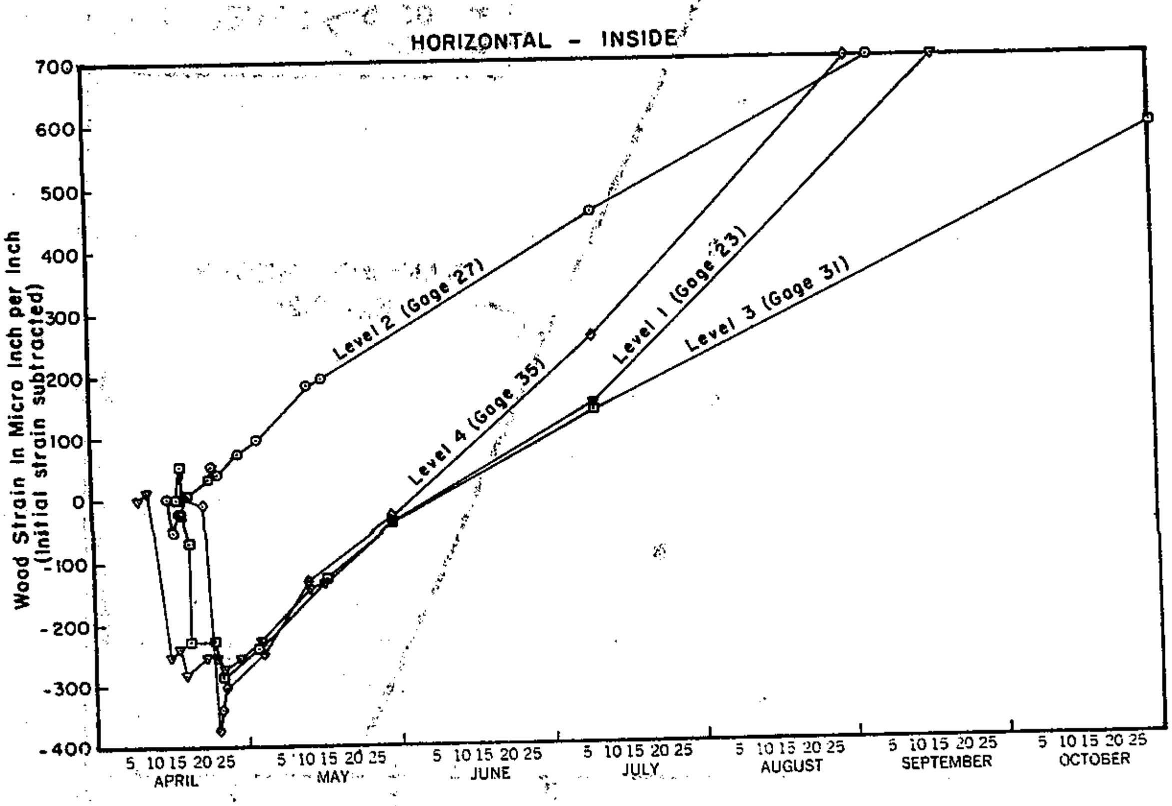



**Fig. 14 PRESSURE CELL READINGS VS OVERBURDEN HEIGHT**

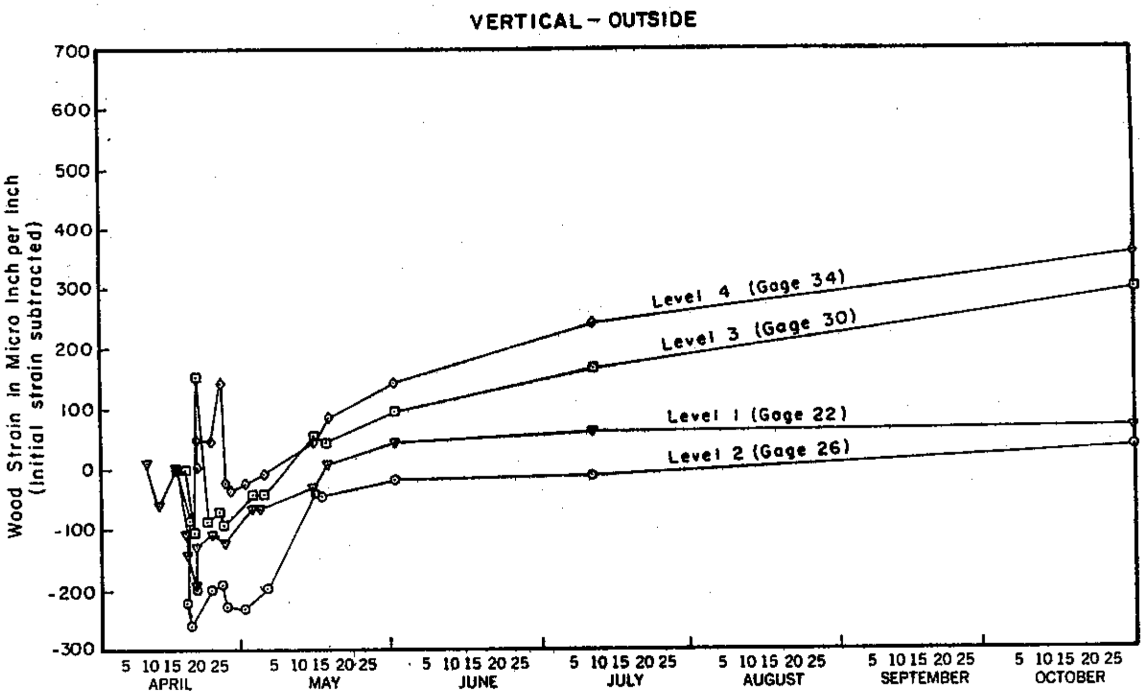



**15a MEASURED STRAIN IN PLYWOOD PANELS vs TIME (Horizontal Direction - Outside Gages)**

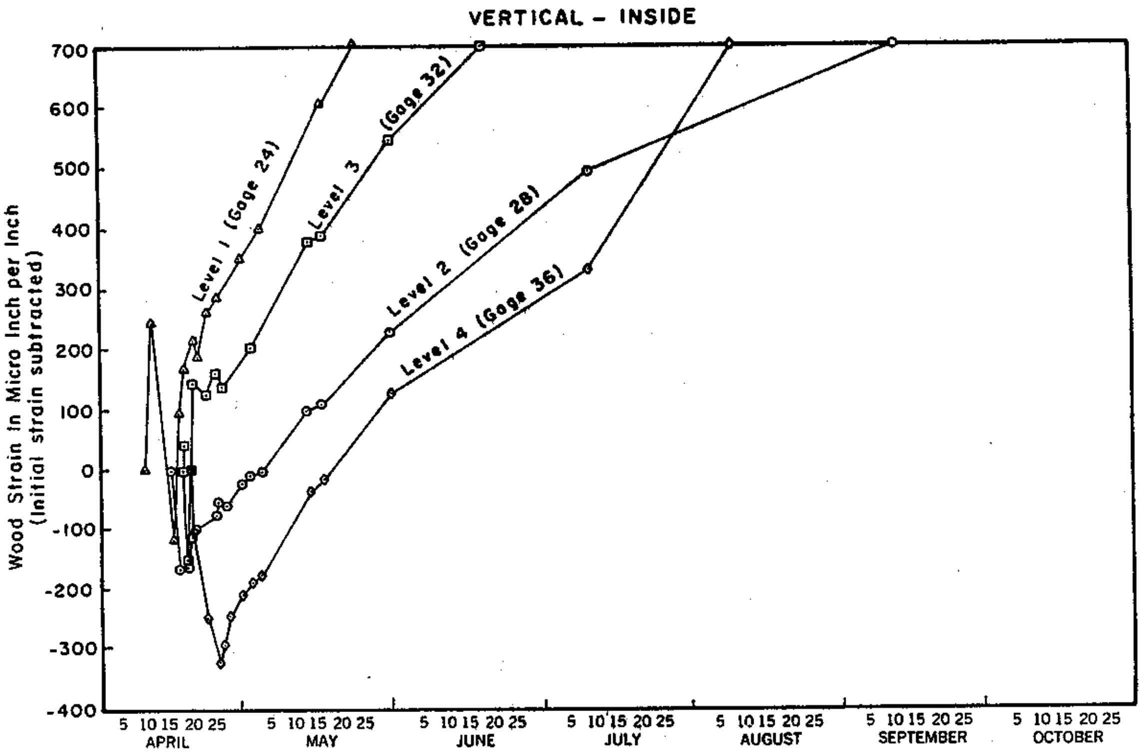



**15b MEASURED STRAIN IN PLYWOOD PANELS vs TIME (Horizontal Direction - Inside Gages)**

**FIGURE 15**




**16a MEASURED STRAIN IN PLYWOOD PANELS vs TIME (Vertical Direction – Outside Gages)**




**16b MEASURED STRAIN IN PLYWOOD PANELS vs TIME (Vertical Direction – Inside Gages)**

**FIGURE 16**

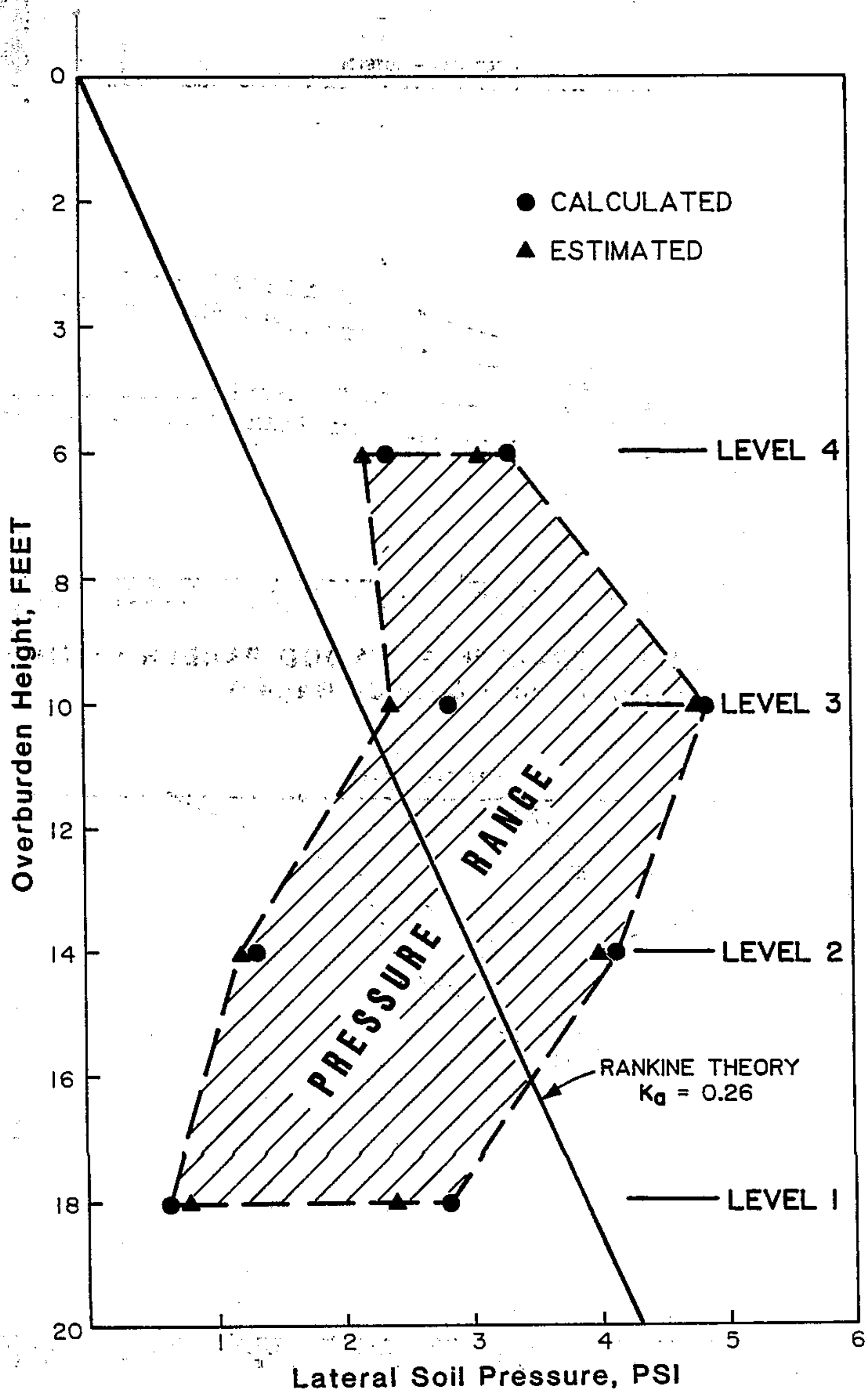



Fig. 17 LATERAL SOIL PRESSURE FROM FACE PANEL STRAIN VS OVERBURDEN

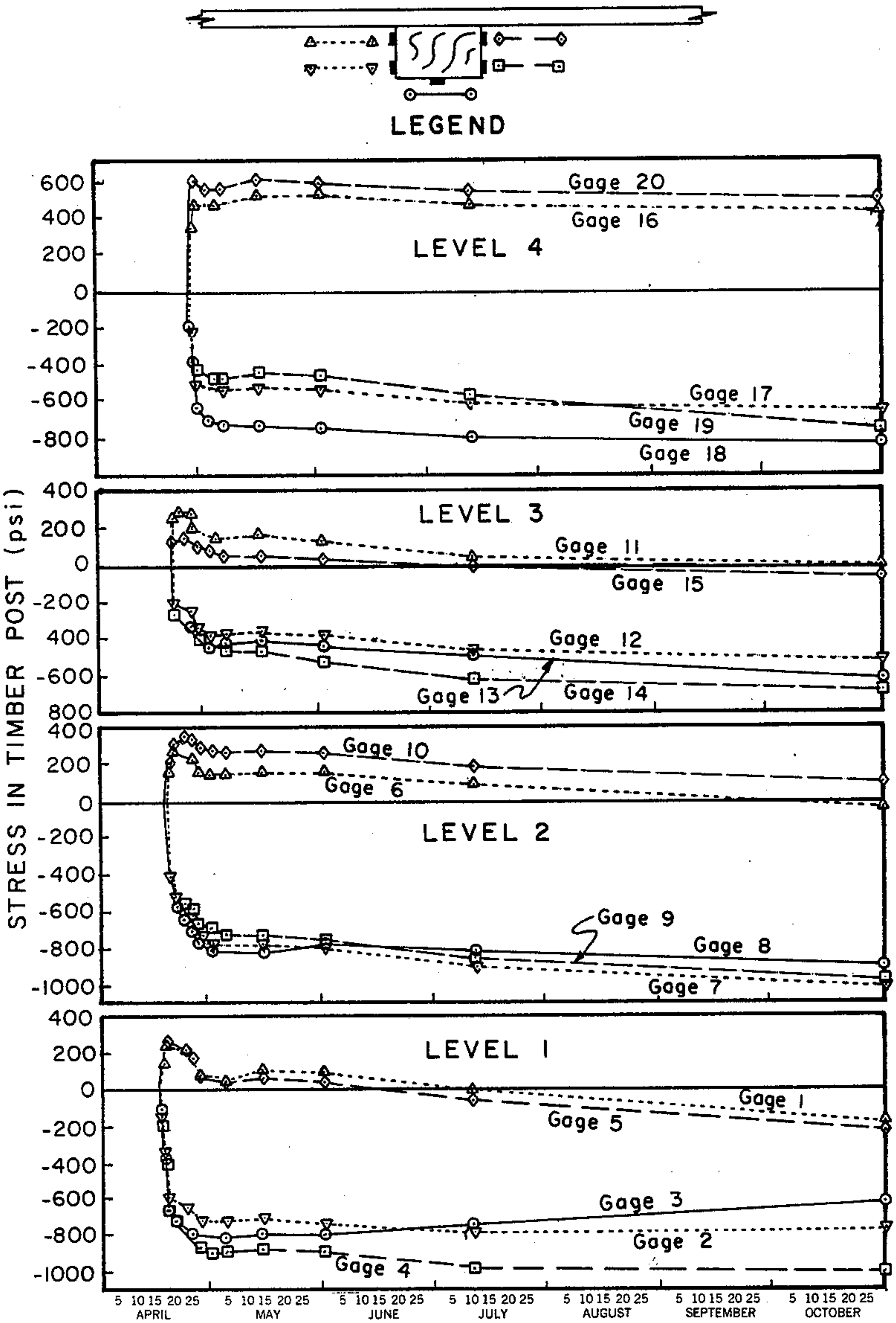



Fig. 18. HISTORY OF STRESSES IN VERTICAL WOOD POSTS

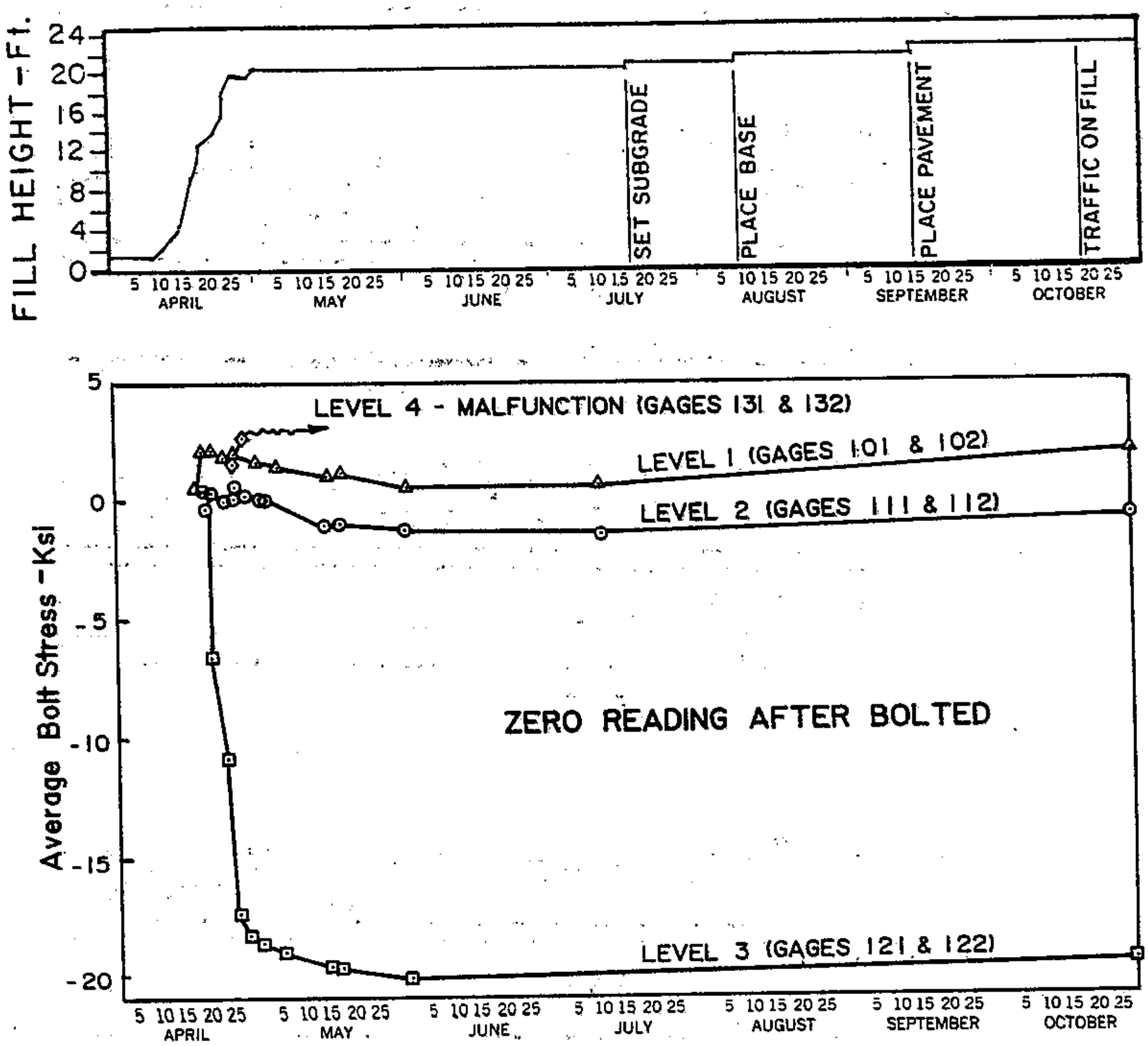



Fig. 19. HISTORY OF STRESSES IN BAR-MAT CONNECTOR BOLTS

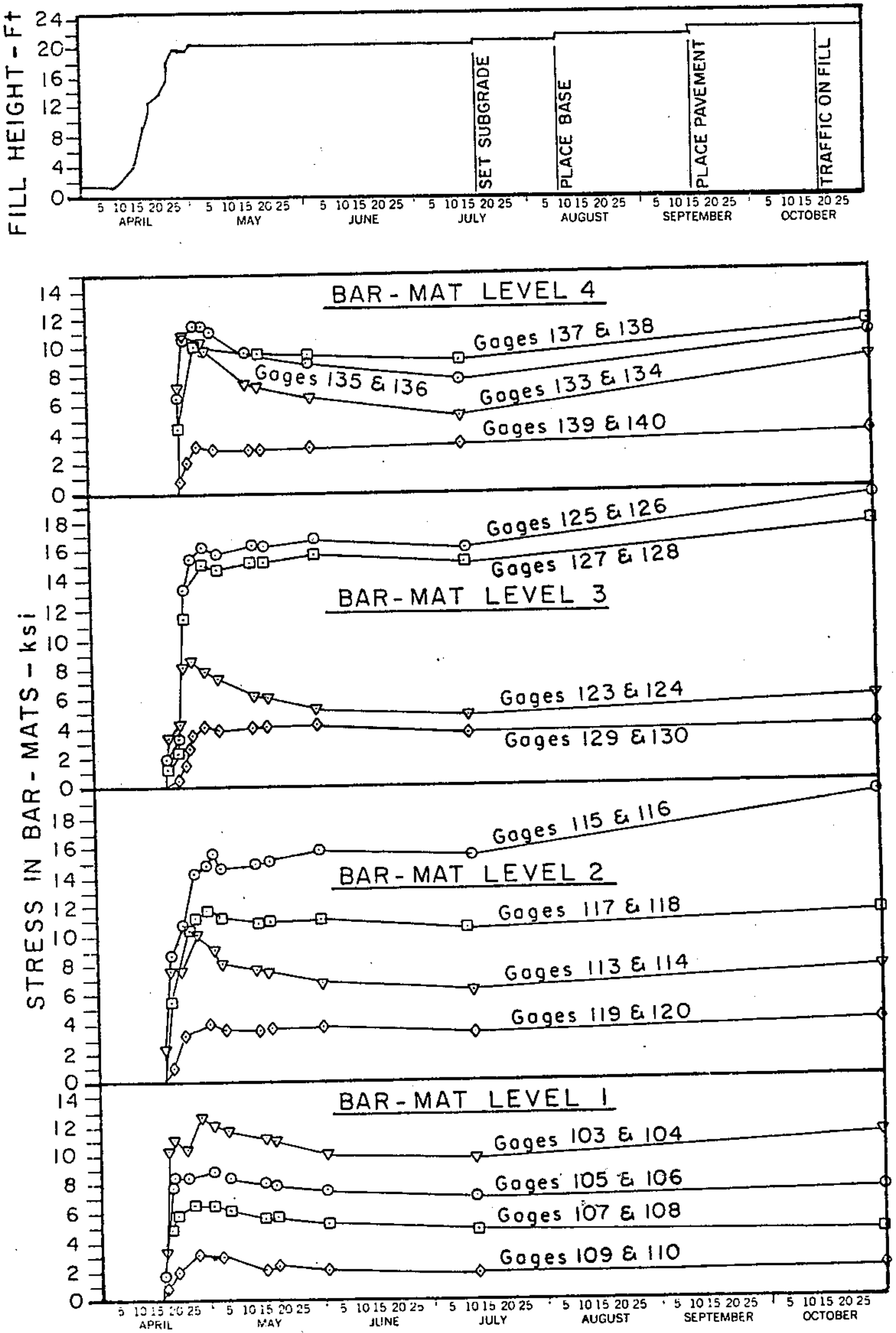



**Fig. 20. HISTORY OF STRESSES IN BAR-MATS**

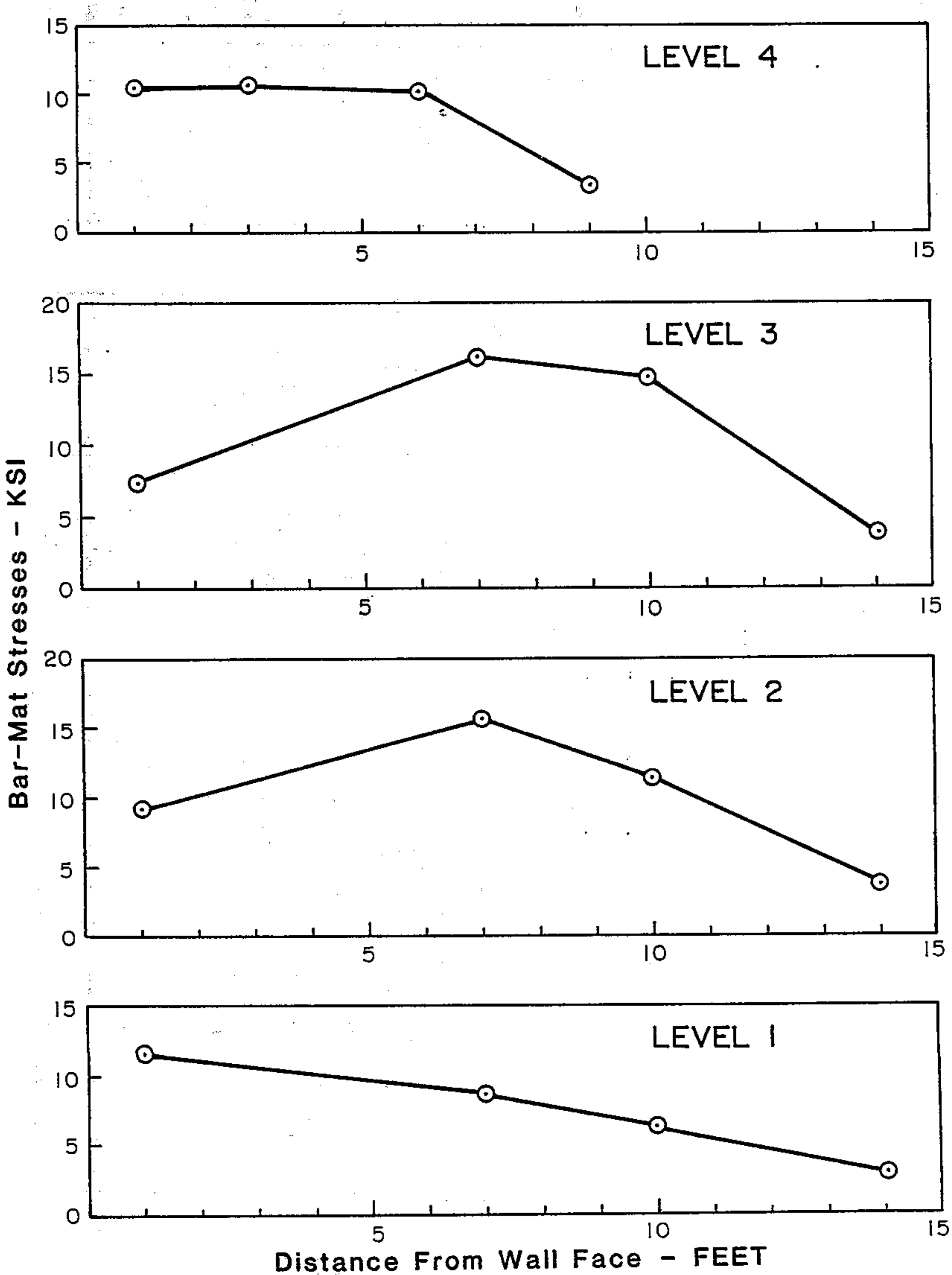



Fig. 21 BAR-MAT STRESSES AT WALL COMPLETION (May 2, 1979)

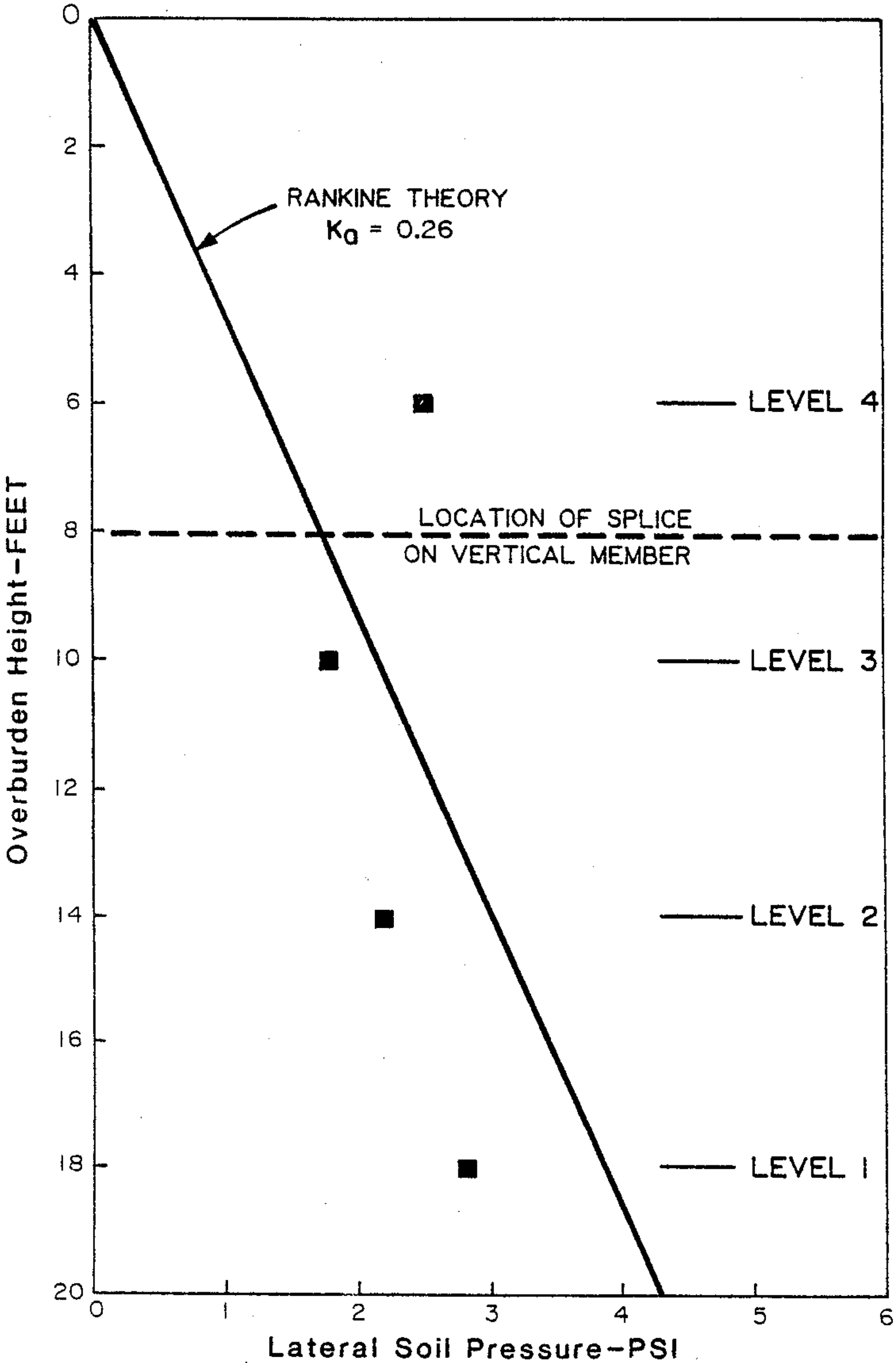



**Fig. 22 LATERAL SOIL PRESSURE DETERMINED FROM BAR-MAT STRESSES ONE FOOT BACK FROM FACE**

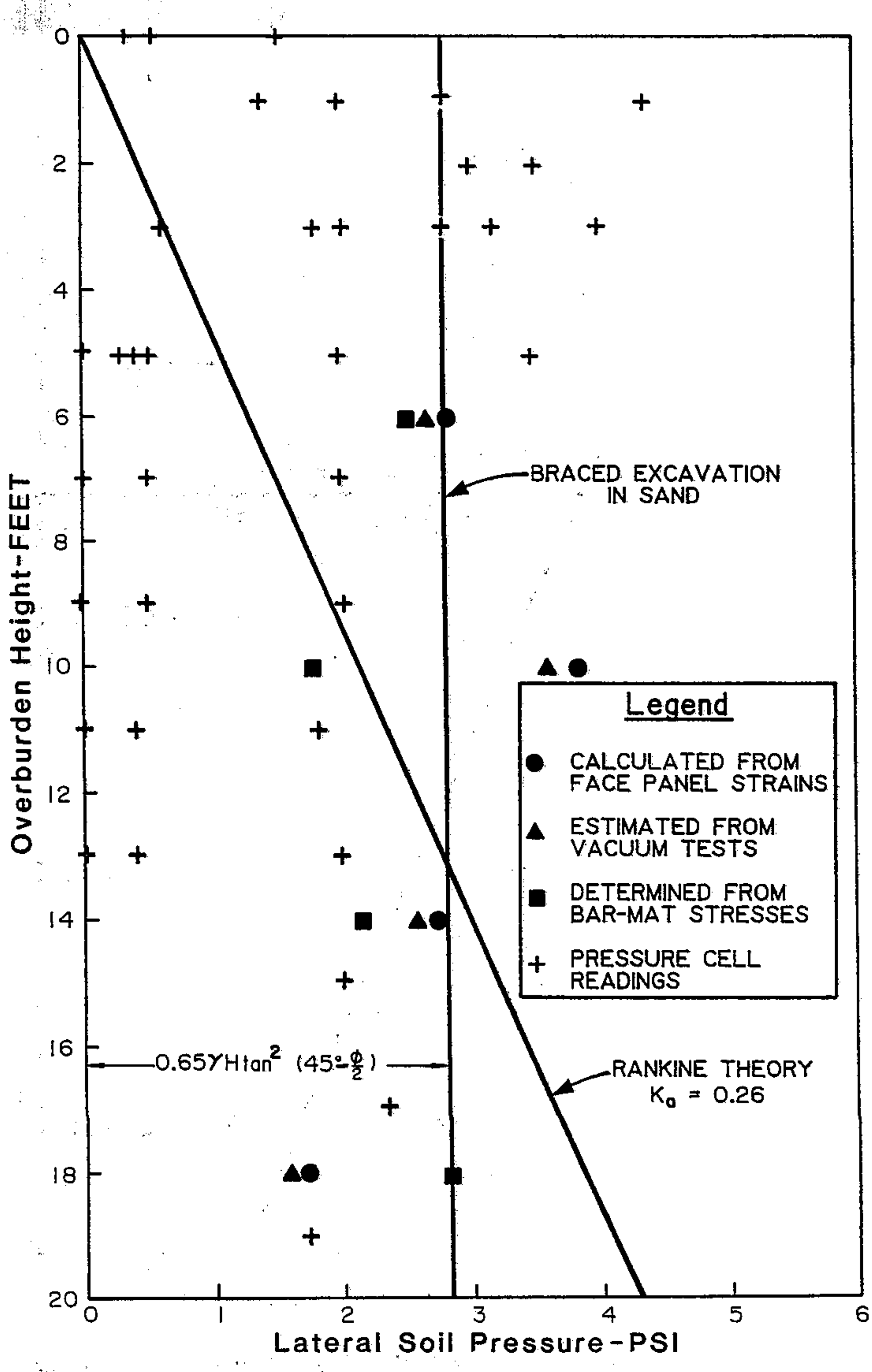



Fig. 23 LATERAL SOIL PRESSURE TEST RESULTS VS THEORY

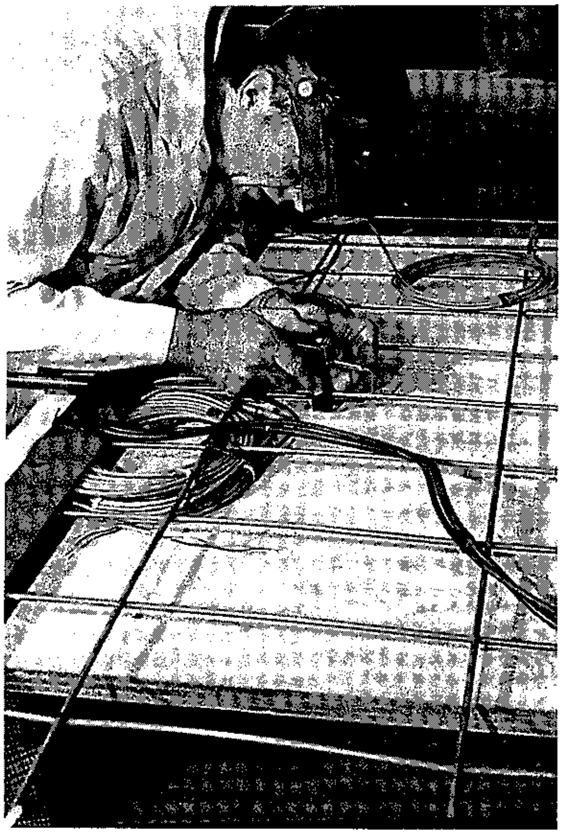


PHOTO 1: Welding of strain gages on bar-mats

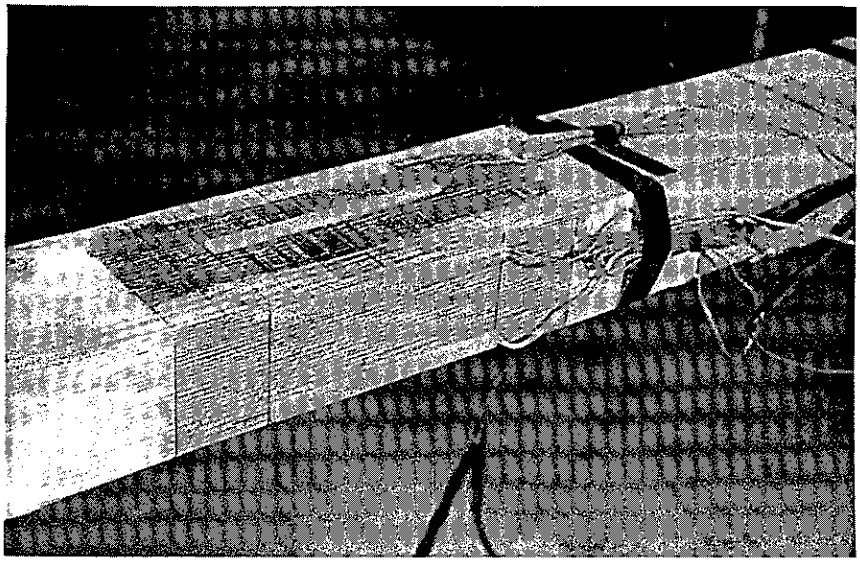


PHOTO 2: Strain gages on wood post

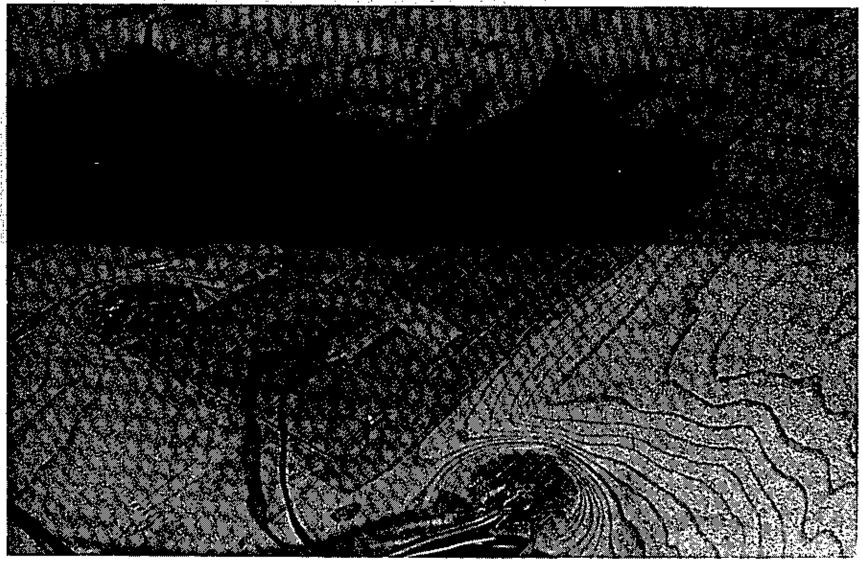


PHOTO 3: Strain gages on plywood panel

PHOTO 4: Laboratory setup for testing timber posts to determine elastic properties.

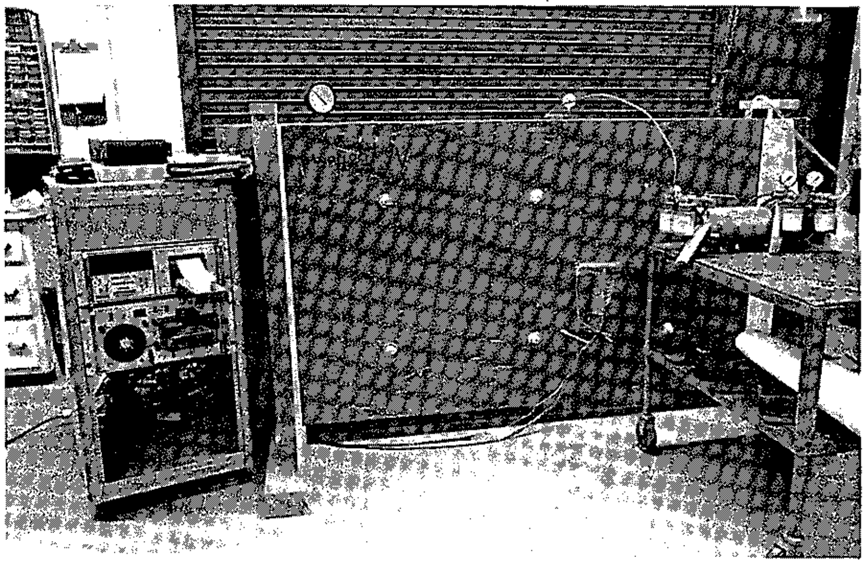


PHOTO 5: Special test frame for vacuum load tests on plywood panels.

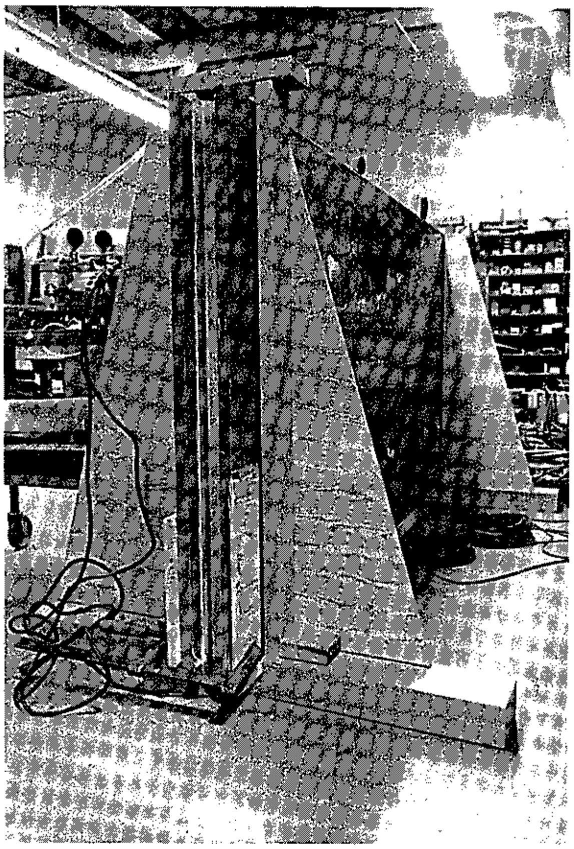


PHOTO 6: Special test frame for vacuum load tests on plywood panels.

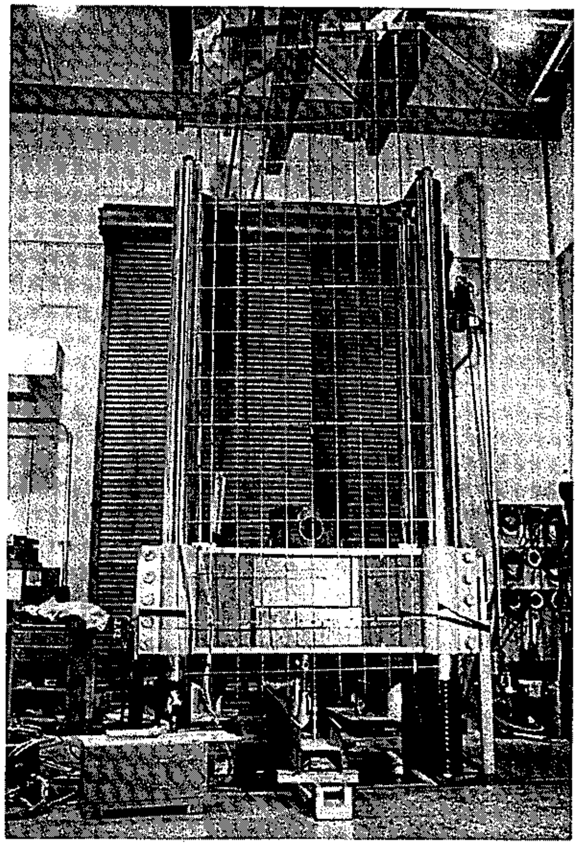


PHOTO 7: Apparatus for testing linearity of strain gages.

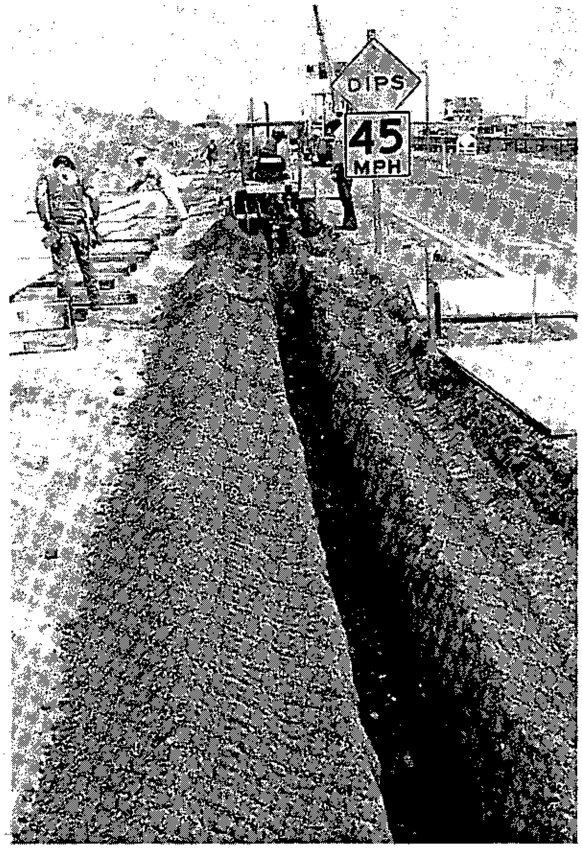



PHOTO 8: Excavation of footing trench

PHOTO 9: Plywood panels nailed on four wooden posts to form a prefabricated unit.

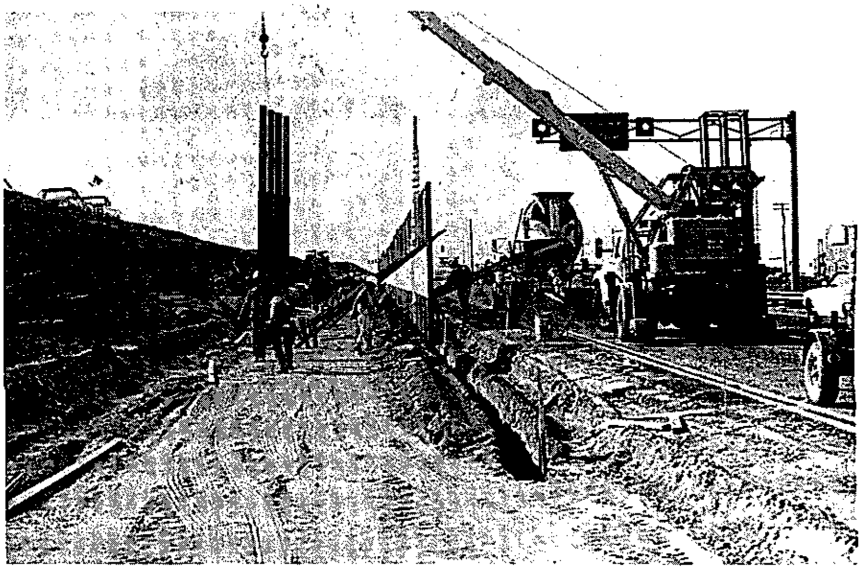
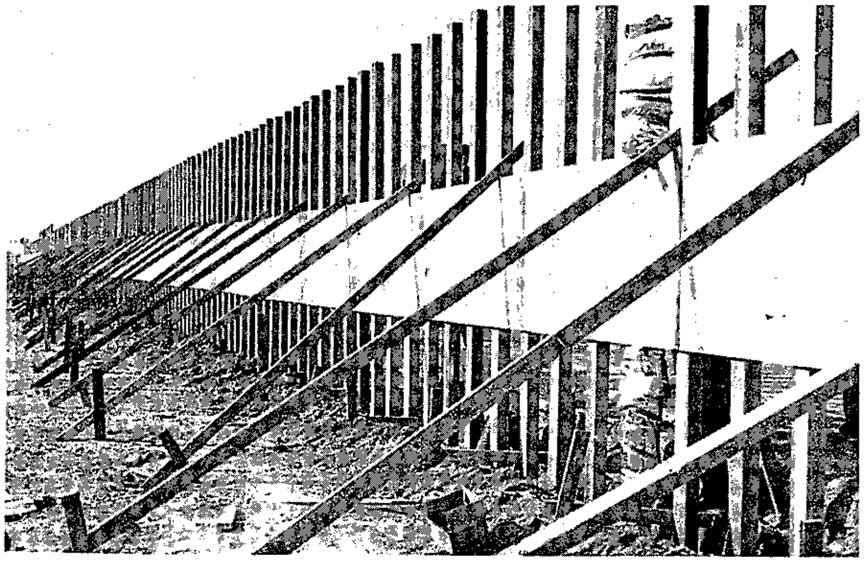
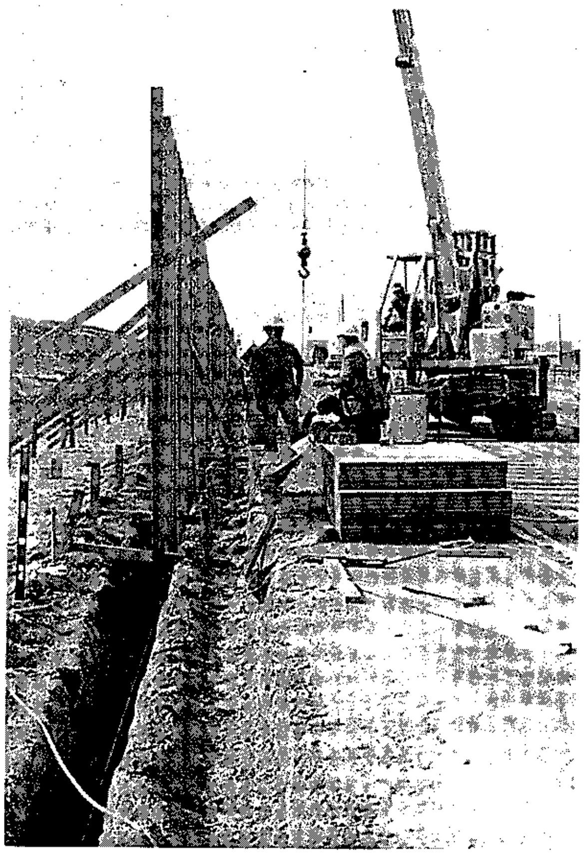


PHOTO 10: Erection of prenailed wooden facing units.





PHOTOS 11 & 12: Lateral supports for wooden facing units.

PHOTO 13: Wood posts embedded in concrete footing.

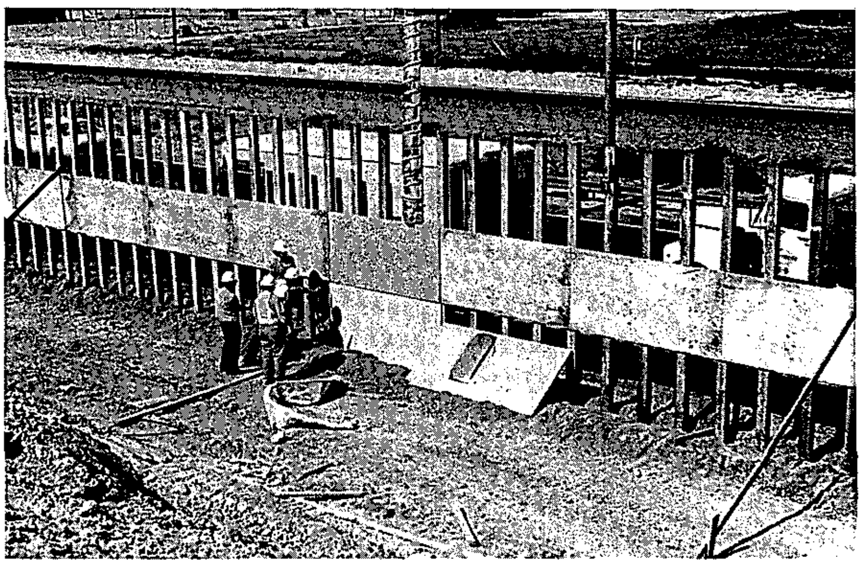


PHOTO 14: Installation of bottom level plywood panels.

PHOTO 15: Drilling bolt holes through plywood panel and wood posts.

PHOTO 16: Placement of bar-mats.

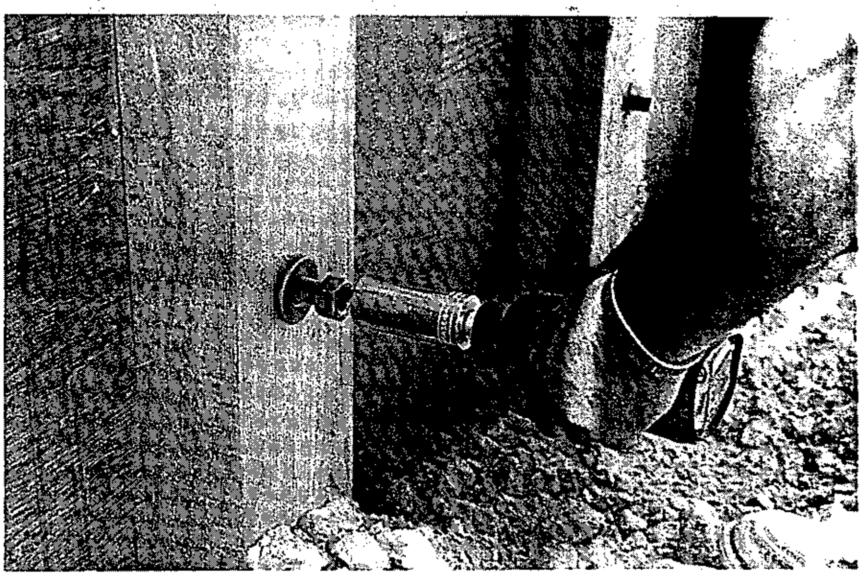


PHOTO 17: Tightening bolts on front face of wall.

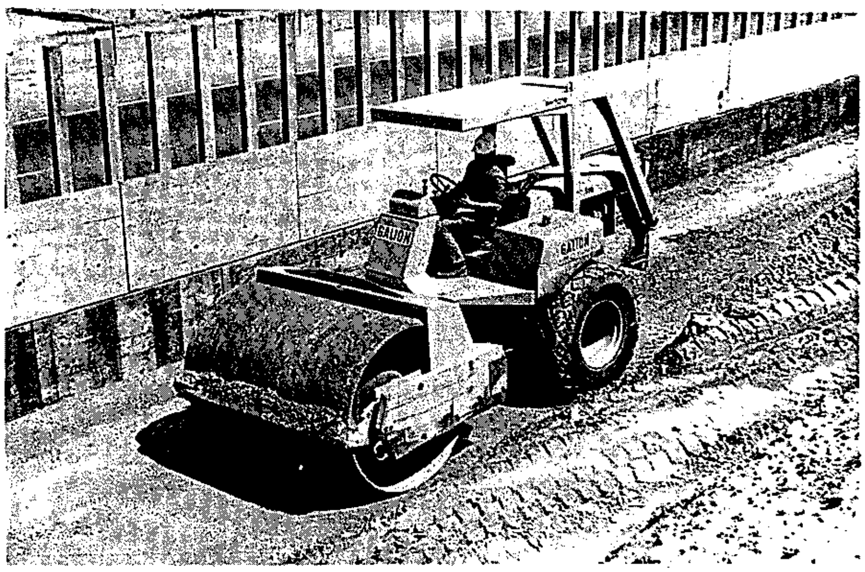


PHOTO 18: Compaction of embankment fill material with a vibratory roller.

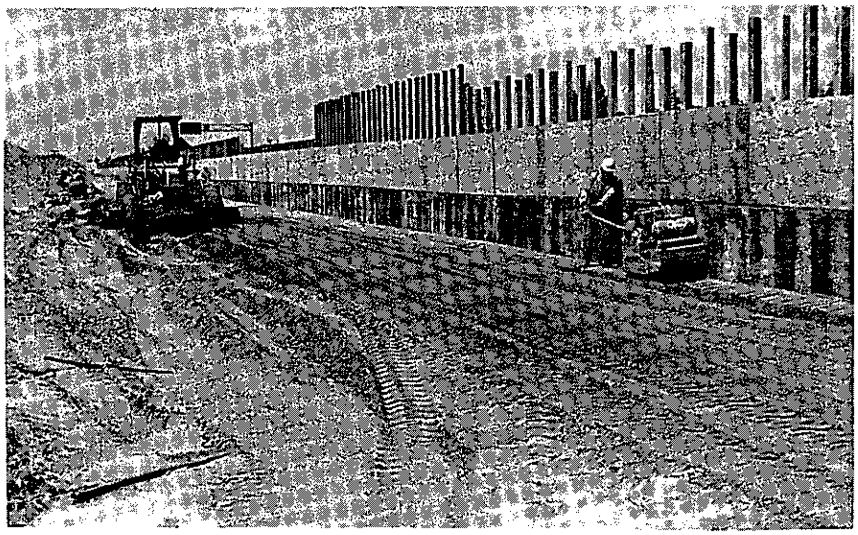


PHOTO 19: Compaction of fill material within two feet of wall face using hand operated vibratory roller.

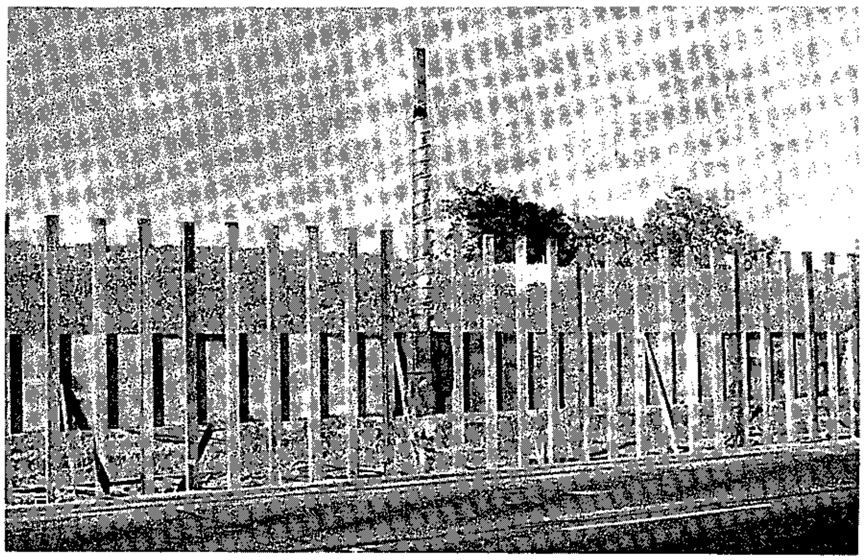


PHOTO 20: Instrumented wood post erected in position.

PHOTO 21: Field installation of soil pressure cells behind wall face.

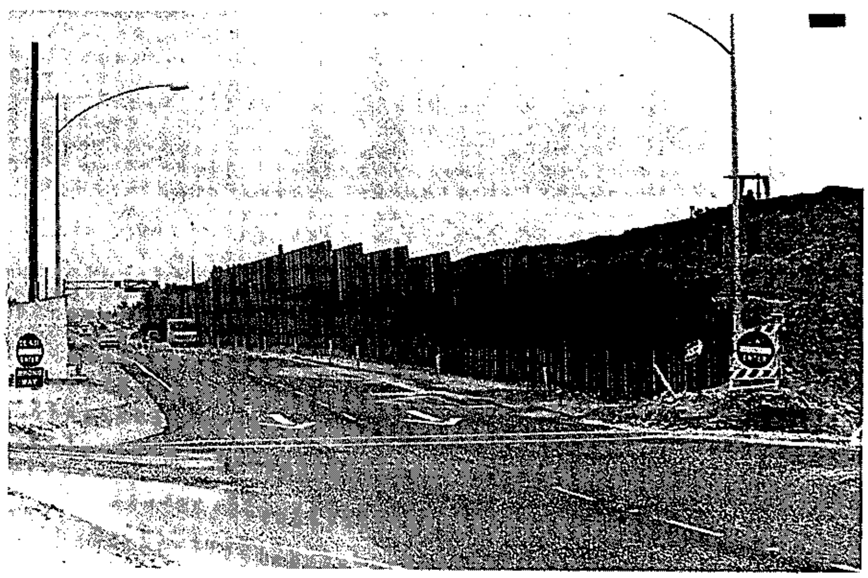


PHOTO 22: Wood-faced MSE nearing completion.

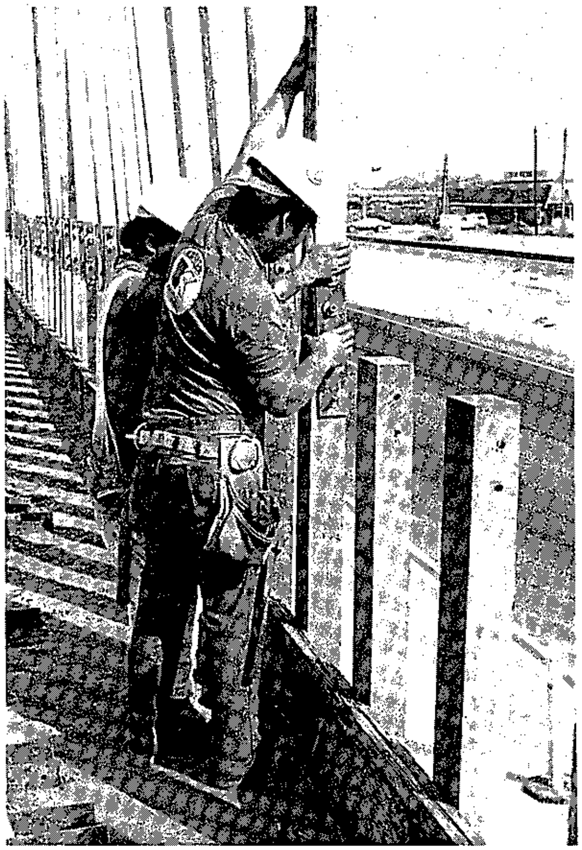


PHOTO 23: Installation of splice on vertical wood posts.

## APPENDIX A

### THEORY FOR DETERMINATION OF MODULUS OF ELASTICITY OF DOUGLAS FIR POSTS

For a simply supported beam undergoing elastic deformation from a concentrated load at any point, classical mechanics of materials analysis yields the following equation:

$$\Delta_a = \frac{Pa^2b^2}{3\ EI\ell} \qquad (1)$$

where $\Delta_a$ = the deflection at point of load
$P$ = applied load
$a$ = shortest distance between support and load
$b$ = longest distance between support and load
$E$ = modulus of elasticity
$I$ = moment of inertia about the u-u axis
$\ell$ = length of the beam

Refer to Figure 8 of text.

When the relationship between P and $\Delta$ is known, E remains the only unknown in Equation (1). The moment of inertia, I, about the u-u axis is found from the equation:

$$I = \frac{wh^3}{12} \qquad (2)$$

For an average beam width (w) of 5.52 in. and average beam depth (h) of 3.81 in.,

$$I = \frac{(5.52)(3.81)^3}{12} = 25.44 \text{ in.}^4$$

In the case of a slightly warped beam (see Figure 8b), P = P (cos$\phi$), therefore Equation (1) becomes

$$E = \left(\frac{Pa^2b^2}{\Delta_a}\right)\left(\frac{\cos\phi}{3I\ell}\right) \quad \dots\dots\dots\dots\dots \quad (3)$$

assuming $\phi = 1°$ for analysis.

From Figure 9, $\frac{P}{\Delta_a} = 117$ lb/in.

From the beam geometry shown in Figure 8,

$$\left(\frac{\cos\phi \, a^2b^2}{3I\ell}\right) = K = 15{,}166/\text{in.}$$

and E is calculated as

$$(117)(15{,}166) \frac{\text{lb}}{\text{in}^2} = 1.77 \times 10^6 \text{ psi}$$

for the instrumented beam.

APPENDIX B

## THEORY FOR DETERMINATION OF MODULUS OF ELASTICITY OF PLYWOOD PANELS

For a simply-supported member with a centrally applied load, classical mechanics of materials analysis yields the relation:

$$E_{11} = \frac{Px}{48\Delta_x I_{11}} (3\ell^2 - 4x^2) \quad \ldots\ldots\ldots\ldots (4)$$

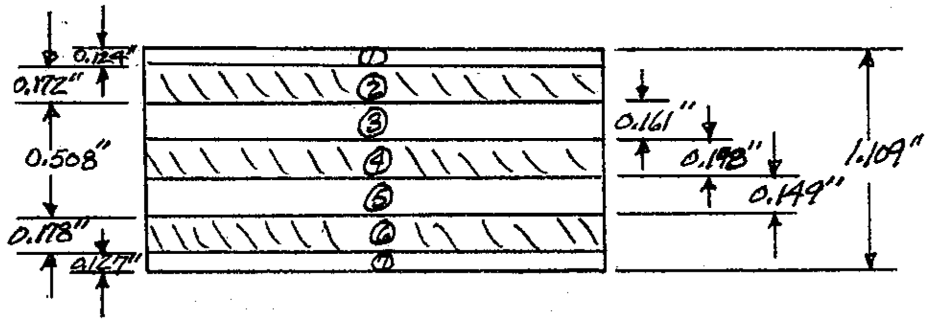
where $P$ = applied load
$x$ = distance from support of measurement point
$\ell$ = length of member
$\Delta_x$ = deflection from the load
$I_{11}$ = moment of inertia parallel to outer grain
and $E_{11}$ = modulus of elasticity parallel to outer grain

Load-deflection relationships, shown in Figure 10 of text, permit solving for $E_{11}$ if $I_{11}$ is known.

The moment of inertia is better understood as a resistance to bending, and is a function of geometry and strength characteristics of the cross section. Due to the makeup of plywood (as described earlier and shown in Figure B-1), the reduced effectiveness of the perpendicularly grained plies to resist bending must be taken into account. The modulus of elasticity of veneer perpendicular to the grain is about 1/35 that of its modulus parallel to the grain (APA 1978). Thus, a ratio of elasticities of the inner ply to the outer

Fig. B-1 PROPERTIES OF PLYWOOD

CROSS-SECTION



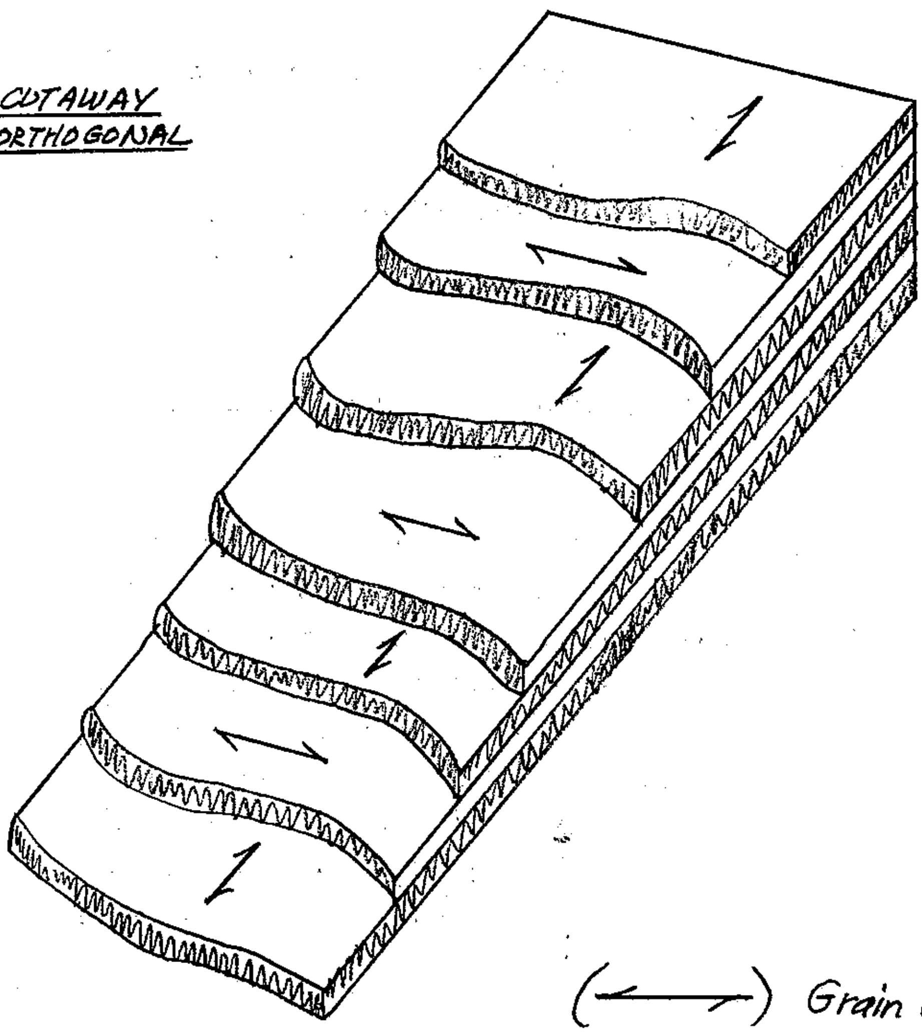

Dimensions shown are in inches and Represent Average Dimensions of Boards Tested.

7 Plies

Average Total width = 1.109 in.

For all plies $\begin{cases} E_{11} \text{ Parallel to grain} = 1.8 \times 10^6 \text{ psi} \\ E \text{ perpendicular to grain} = \frac{1}{35} E_{11} \end{cases}$

CUTAWAY ORTHOGONAL



(⟵⟶) Grain direction of Ply.

plies is used to calculate the moment of inertia. This idea is conveyed in Figure B-2 where the cross section shown is formed by proportioning the stiffness of each layer relative to the stiffness of the outer layer. The moment of inertia of each individual ply about its center, $I_{oi}$, is calculated and $I_{11}$, the moment of inertia of the entire cross section about its neutral axis, is calculated by summing up the moments of inertia of each ply about the outer surface of the cross section using the parallel axis theory.

$$I_{11} = \sum_{i=1}^{n=7} [I_{oi} + A_i(\overline{y} - \overline{y}_i)^2 \quad \ldots\ldots\ldots\ldots \quad (5)$$

where $I_{oi}$ = moment of inertia of a ply abouts its centroidal axis

$A_i$ = area of ply (i) in the proportioned cross section

$\overline{y}$ = distance from centroid of proportioned cross section to top surface of cross section

$y_i$ = distance from centroid of ply (i) to top of surface of cross section

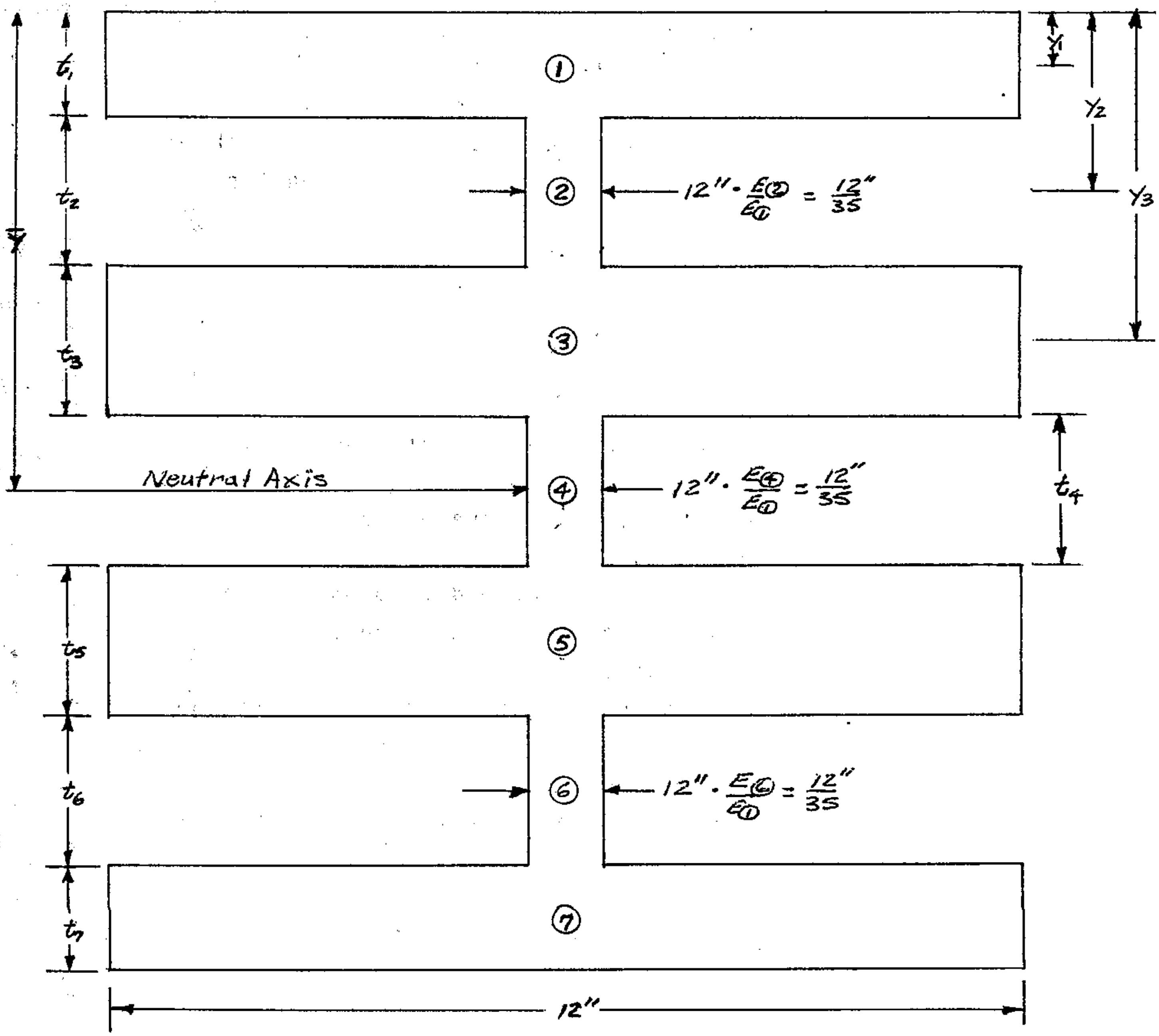
For panel A, $I_{11}$ = 0.8916 $in^4$ per foot of cross sectional width.

With $I_{11}$ known, $E_{11}$ is found using Equation (4) at $\Delta$=1.0 inch.

For panel A,

$$E_{11} = \frac{(443.63\ \text{lb/in.})(36\ \text{in.})[3(96)^2 - 4(36)^2](\text{in.}^2)}{48(0.8916)(\text{in.}^4/\text{ft})(4)(\text{ft})} = 2.1 \times 10^6\ \text{psi}$$

Fig. B-2 – PROPORTIONED CROSS-SECTION FOR MOMENT OF INERTIA CALCULATION




Odd numbered plies are parallel to loading stress

Even numbered plies are perpendicular to loading stress

$\bar{y}$ is found from $\frac{\Sigma yA}{\Sigma A} = \bar{y}$

Modulus of Elasticity is the same for each ply parallel to grain.

## Theoretical Calculation of Strains

With $I_{11}$ known, the effective section modulus (Schuster, 1973), KS, can be found. For plywood, the effective section modulus is defined at $KI_{11}/C$ where K is an empirical correction factor equal to 0.85 and C is the distance from the centroid to the outer surface. KS is found per foot of cross section.

The stress, $\sigma$, in the member is found (Seely, 1952) for the 4 ft cross section from the flexure formula

$$\sigma = \frac{M}{4KS} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where M is the bending moment at the strain gage. The section modulus is multiplied by 4 ft to represent the entire width of the member. The loading diagram is shown in Figure 10 of text and the bending moment at the strain gage location is found from Equation (7) below.

$$M = \frac{Px}{2} + \frac{wx}{2}(\ell - x) \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

where
- $P$ = applied line load at center of span
- $x$ = distance from support to strain gage
- $w$ = weight/ft of panel
- $\ell$ = distance between supports

From the values in Figure 10

$$M = 18P + 1080W \text{ lb-in.} \quad \ldots\ldots\ldots\ldots \quad (8)$$

Substitution into the flexure formula yields:

$$\sigma = \frac{4.5P+270W}{KS} \text{ psi}$$

with σ known, the theoretical strain is found using

Hooke's Law, $\varepsilon = \frac{\sigma}{E}$ . . . . . . . . . . . . . . (9)

APPENDIX C

## 1. THEORETICAL CALCULATION OF STRAINS IN PLYWOOD PANELS FROM VACUUM TEST LOADING

It was assumed that each section of the plywood panel containing the strain gages acts independently. The 4 ft span will be referred to as Span "a". The 2 ft span will be called Span "b".

It was assumed that for bending in Span "b", the plywood panel would act as a 2-ft long beam, 4 ft wide with fixed ends. The strain at the gage location can be determined by combining Equations 6 and 9 from Appendix B to form the following:

$$\varepsilon = \frac{M}{4KSE} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

where $$M = \frac{wb^2}{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

KS = Section modulus from Table 3 (text)
E = Modulus of elasticity from Table 4 (text)
w = Vacuum load in psi

Since 1 inch of Hg = 0.491 psi, the maximum vacuum pressure of 18 inches of Hg is equivalent to 8.84 psi. For a 4 ft wide beam, this corresponds to a vacuum load of 8.84(4)(12) = 424 lb/in.

For the two panels which were tested, one obtains:

$$\varepsilon_{\text{panel A}} = \frac{-(424)(24)^2/(24)}{4(1.74)(2.10\times10^6)} = -696\mu \text{ in./in.}$$

$$\varepsilon_{\text{panel B}} = \frac{-(424)(24)^2/24}{4(1.27)(2.21\times10^6)} = -906\mu \text{ in./in.}$$

For bending in the vertical direction, (Span "a"), the instrumented section of the plywood panel was assumed to act as a flat plate uniformly loaded with Span "b" fixed and Span "a" simply supported. Again, using Equations 6 and 9, the following is obtained:

$$\varepsilon = \frac{M}{2KSE} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

$$M = \frac{wb^2}{80}\left[1+0.3(\alpha)^2\right] \quad \text{(p.236, Table 9, Seely \& Smith, 1952)} \quad . \quad (13)$$

where
- $M$ = Centerline moment
- $\alpha$ = b/a, b=short span, a=long span
- $w$ = Vacuum load in psi
- $E$ = Modulus of elasticity from Table 3 (text)
- $KS$ = Section modulus perpendicular to the direction of the grain (0.82 in.$^3$/ft)

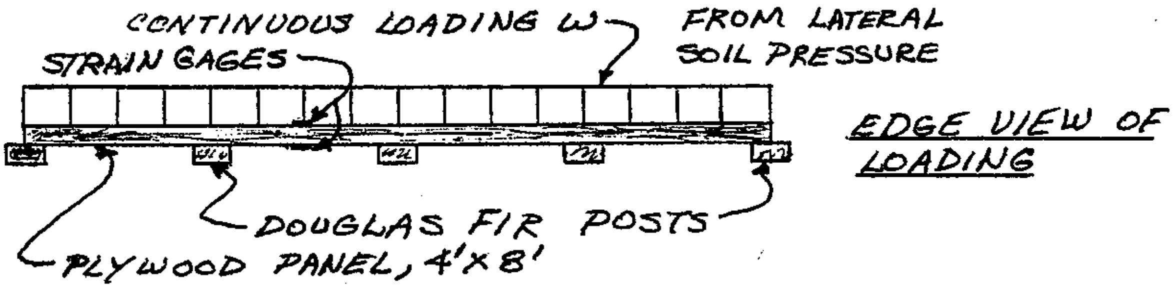
Calculating the theoretical strain in the two panels we see that

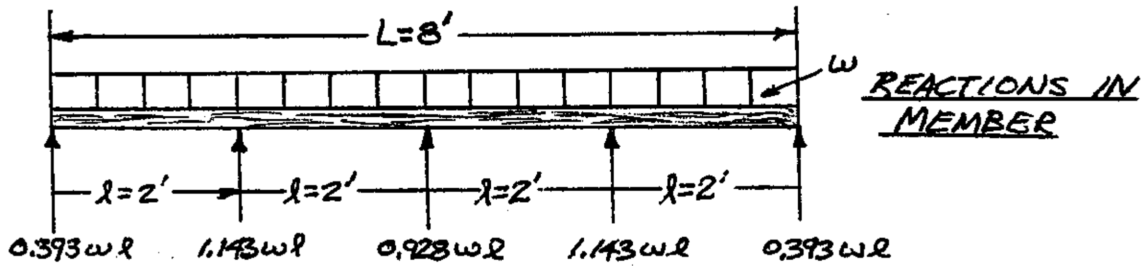
$$\varepsilon'_{\text{panel A}} = \frac{(-8.84)(24)^2/80[1+0.3(1/2)^2]}{2(0.82)(2.10\times10^6)} = -20\mu \text{ in./in.}$$

and,

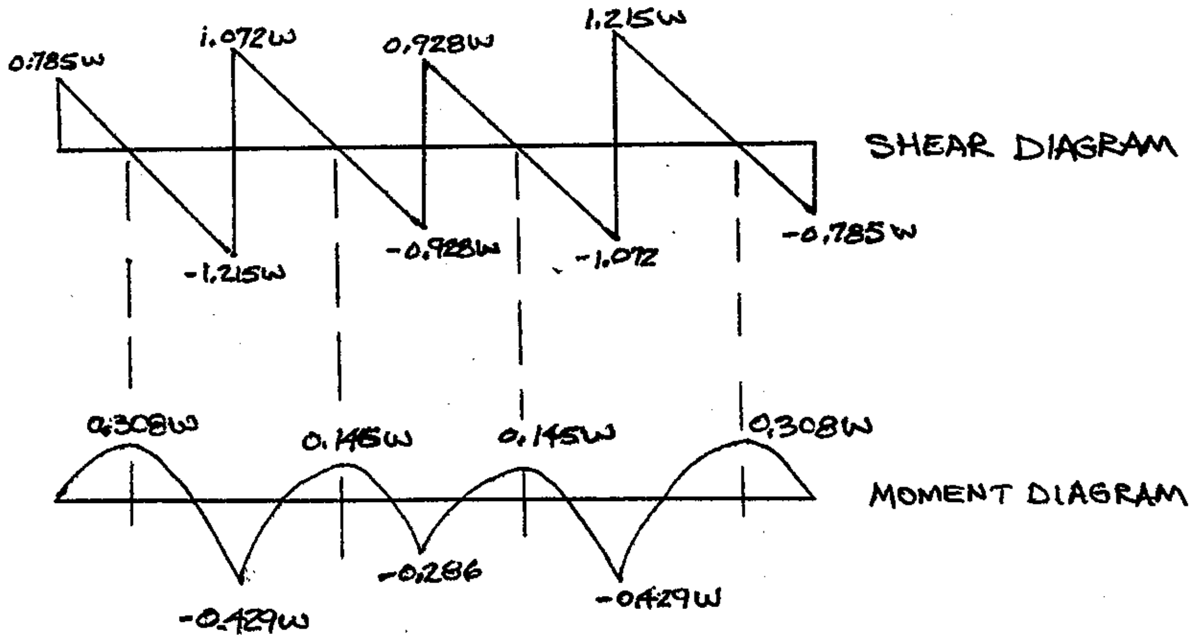
$$\varepsilon'_{\text{panel B}} = \frac{(-8.84)(24)^2/80[1+0.3(1/2)^2]}{2(0.82)(2.21\times10^6)} = -19\mu \text{ in./in.}$$

2. CONTINUOUS BEAM ASSUMPTION FOR DETERMINING SOIL PRESSURE ON WALL FACE USING STRAIN GAGE DATA




The following reactions and bending moments were derived by moment distribution assuming a statically indeterminate beam (refer to Wang, 1953):



## Analysis of Stress and Unit Soil Pressure

From the flexure formula

$$\sigma_{max.} = Mc/I \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (14)$$

For plywood $S \neq I/c$, therefore, use a factor K to correct equation (14) which then becomes

$$\sigma = M/KS \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (15)$$

Combining (10) with (9) we obtain

$$\varepsilon = M/KSE$$

where $\varepsilon$ = measured panel strain in horizontal direction

For two panel thicknesses of plywood, the unit strain can be determined for single panel plywood face unit by the following:

$$\varepsilon_1 = 4\varepsilon_2 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (16)$$

where $\varepsilon_1$ = strain measured on single panel face unit
$\varepsilon_2$ = strain measured on double panel face unit

Levels 1, 2 and 3 are two panels thick.

For Panel A at Level 1 (from strain gage 21), the average unit soil pressure is derived as follows using KS and E values from Table 3.

$$\varepsilon = M/KSE = \frac{20.736\ W\ lb/in.}{1.738\ in.^3/ft(4\ ft)2.0957x10^6 lb/in.^2}$$

$$\varepsilon = 1.423x10^{-6} W$$

$$W = \varepsilon/1.423x10^{-6}(48\ in.) = 1.464x10^4 \varepsilon lb/in.^2$$

Since panels are doubled, use $\varepsilon_1 = 4\varepsilon_2$ and measured strain at gage 21, $\varepsilon_2 = 48x10^{-6}$ in./in.

$$W = 1.464x10^4(48x10^{-6})4 = 2.81\ psi$$

Refer to Table 7

For single panel units, i.e., Level 4 Panel F, use $\varepsilon_1$ without modification and measured strain at gage 33, $\varepsilon_1 = 220x10^{-6}$ in./in.

$$\varepsilon = \frac{20.736\ W}{1.412(4)(1.909x10^6)} = 1.923x10^{-6} W$$

$$W = 2.38\ psi$$

## APPENDIX D

### THEORETICAL CALCULATION OF STRAINS IN WOOD POSTS FROM LABORATORY LOADINGS

The beam obtained for instrumentation was somewhat warped due to shrinkage and could induce torsional stresses during loading through unsymmetrical bending. However, the angle of warp was found to be minimal (not greater than 2°) and torsional stresses could, therefore, be neglected in the analysis. The shear stress due to the dead load (beam exclusive of bending) was also insignificant compared to the bending stress and was neglected.

Given the geometry of the beam, the bending stress for unsymmetrical loading is given (Seely and Smith, 1952) by

$$\sigma = \frac{(M\cos_{\phi})v}{I_u} \pm \frac{(M\sin_{\phi})u}{I_v} \quad \ldots\ldots\ldots\ldots \quad (17)$$

where $\sigma$ = bending stress

$M$ = bending moment

$\phi$ = warp angle

(see Fig. 8) $V$ = vertical distance from principal axis of inertia

$u$ = horizontal distance from principal axis of inertia

$I_v$ = moment of inertia about vertical axis

$I_u$ = moment of inertia about horizontal axis

$\sigma$ is positive if the member is in tension and negative if in compression. After the bending stress, $\sigma$, is calculated, the theoretical strain, $\varepsilon$, is found from Equation (9)

where $\varepsilon = \frac{\sigma}{E}$