DESIGN **G**UIDELINE

DG 453
Field Design Standards
Issue No. 4

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Approved By:
Frank Mondello, P.E., Chief Structural Engineer

Issue Record

No	Date	Description of Change	Entered By	Formal Review	Intermediate Review
0	12/15/86	Initial Issue	Khan	x	
1	08/04/94	Reissue of guideline dated December 1985, reprinted July, 1993	Khan	x	
2	03/12/98	Reissue in MS Word formatted	Kenkre		x
3	08/12/04	Added reference to DG452A	McMorrow		x
4	12/29/06	Reissue of existing guideline – 5 yr formal review interval	Kenkre	x	
	05/28/14	Madan Naik's Title Change			

Division of Engineering Services
Madan Naik, P.E.
Vice President and Deputy Chief Engineer

FOREWORD:

MTA New York City Transit is, as a minimum, required to comply with the provisions of the Building Code of New York State. In addition to the above, applicable provisions of the following Design Guidelines (DG) must be considered on all projects:

- **DG 453 Field Design Standards**

  This guideline provides information relating to the design of temporary support systems, underpinning, decking, tiebacks and utility support. This document is primarily intended for use on projects involving modifications and/or rehabilitation of existing transit structures or facilities. The requirements of Section UP for underpinning shall apply to all projects that utilize a Flexible Wall System for lateral support of the excavation.

- **DG 452A Structural Design Guidelines**
  Subway and Underground Structures

  This guideline provides information relating to loads and in addition, design criteria are provided for subway structures. This document is primarily intended for use on projects involving construction of new below ground transit structures or facilities. The requirements of Chapter 11 for underpinning shall apply to all projects that utilize a Rigid Wall System for lateral support of the excavation.

Each project is to be evaluated at the initiation of design to determine its applicability to the provisions of each of the aforementioned Design Guidelines.

MTA NEW YORK CITY TRANSIT

DEPARTMENT OF CAPITAL PROGRAM MANAGEMENT

DIVISION OF ENGINEERING SERVICES

CIVIL/STRUCTURAL

FIELD DESIGN STANDARDS

December 1986

FIELD DESIGN STANDARDS

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SECTION RS

REFERENCE STANDARDS

REFERENCE STANDARDS

The materials, design, and construction methods to be utilized for Field Design Structures shall meet the requirements of the following reference standards:

Reference Standard RS-1, Manual of Steel Construction (AISC), 7th Edition, 1970.

Reference Standard RS-2, Manual of Steel Construction (AISC), 6th Edition, 1963. (Note: This reference standard is to be applied only to A-7 steel).

Reference Standard RS-3, New York City Transit Authority Structural Design Standards, 1973.

Reference Standard RS-4, National Design Specifications for Stress-Grade Lumber and its Fastenings, National Forest Product Association, 1968.

Reference Standard RS-5, "Physical, Stress-strain, and Strength Responses of Granular Soils", Donald M. Burmister, Special Technical Publication No. 332 (ASTM), 1962.

Reference Standard RS-6, Design Manual - Soil Mechanics, Foundations and Earth Structures, Navdocks DM-7, (Gov't. Printing Office), March 1971.

Reference Standard RS-7, Soil Mechanics in Engineering Practice, K. Terzaghi and R.B. Peck, 2nd Edition, John Wiley and Sons, Inc., New York, 1967.

Reference Standard RS-8, Steel Sheet Piling Design Manual, U.S. Steel, ADUSS 25-3848-03, April 1972.

Reference Standard RS-9, The City of New York Building Code as amended August 22, 1969.

Reference Standard RS-10, Rock Mechanics in Engineering Practice, Editors M. Statt and O.C. Zienkiewicz, John Wiley and Sons, Inc., New York, 1968.

Reference Standard RS-11, Pipe Piles, Properties and Dimensions for Designing, AISU, April 1971.

Reference Standard RS-12, Tiebacks in Foundation Engineering and Construction, Harry Schnabel, Jr., McGraw-Hill, Inc., New York, 1982.

SECTION AS

ALLOWABLE UNIT STRESSES

FOR

TEMPORARY STRUCTURES AND UNDERPINNING

ALLOWABLE UNIT STRESSES FOR TEMPORARY STRUCTURES AND UNDERPINNING

The allowable unit stresses of steel, concrete and lumber to the used for the design of temporary structures and underpinning shall meet the requirements of the following reference standards:

<u>Structural Steel for Temporary Structures and Underpinning of Buildings</u>

Reference Standard RS-1, Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings.

<u>A-7 Structural Steel for Temporary Structures</u>

Reference Standard RS-2, Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

<u>Structural Steel for the Support and Underpinning of Transit Authority Railroad Structures</u>

Reference Standard RS-3, Allowable Unit Stresses (T.A.) for Structural Steel.

<u>Reinforced Concrete</u>

Reference Standard RS-3, Design of Concrete and Reinforced Concrete Structures.

<u>Lumber</u>

Reference Standard RS-4, Allowable Unit Stresses - "Engineered Uses".

<u>SPECIAL PROVISIONS</u>

1. All new steel for temporary structures and underpinning shall be ASTM A36. Other grades of steel may be substituted only with the approval of the Engineer.

2. If new steel is used for temporary structures, the allowable unit stresses indicated in Reference Standard RS-1 may be increased by 20% except for columns and struts or as otherwise noted in these standards or as directed by the Engineer. New steel is defined as steel being used for the first time or steel that had first been used on a previous Authority contract and is being reused after an inspection by the Engineer has found it to be in good condition. No increase in allowable stresses will be permitted for new steel used for temporary structures within the Railroad.

3. Used steel for temporary structures and underpinning will be permitted subject to the Engineer's approval of the material. The unit stresses for used steel shall not exceed the allowable unit stresses indicated in reference RS-1 or, in the case of A-7 steel, reference RS-2. Used steel used for temporary structures or underpinning of Authority structures shall not exceed the allowable unit stresses indicated in Reference Standard RS-3.

4. No increase in allowable unit stresses will be permitted. New steel is to be used in underpinning structures, using allowable unit stress indicated in Reference Standard RS-3.

5. For primary and secondary bracing members, soldier beams, struts, etc., subjected to axial loads only, or to combined axial loads and flexure, no increase in allowable unit stresses will be permitted for the axial compression portion of the load.

6. Secondary bracing shall be designed using a minimum axial load of 2% of the load in the primary bracing member. Secondary bracing is defined as bracing that is required to reduce the unbraced length in either the major or minor axis, of the primary bracing members. The unit stresses shall not exceed the allowable unit stresses given in the appropriate reference standards.

7. Unit stresses for stress field welded connections shall not exceed 75% of the allowable unit stresses indicated in Reference Standard RS-1.

8. Field welded connections will be permitted for underpinning structures. Approval to use field welded connections for underpinning will be granted on an individual basis by the Engineer.

9. For new cold worked curved steel tunnel roof supports in rock tunnels, the design maximum allowable fiber stress shall not exceed 27,000 psi. For new straight steel tunnel roof supports (not cold worked), the design maximum allowable fiber stress shall not exceed 24,000 psi. For steel columns, the unit stresses shall not exceed the allowable unit stresses indicated in RS-1.

10. For new horizontal timber sheeting subjected to the loading indicated in the Field Design Standards for Lateral Earth Pressure Distribution for Temporary Earth Retaining Structures, the allowable unit stresses indicated in Reference Standard RS-4 may be increased by 50% except as otherwise noted in these standards or as directed by the Engineer. No increase in allowable unit stress will be permitted for used horizontal timber sheeting unless inspected by the Engineer and determined to be in good condition. If sheeting is to be left in place, then it must be treated.

11. Unit stresses and general design guide lines not given in the Field Design Standard or the Reference Standards should be submitted to the Authority for approval before their use in the preparation of working drawings.

12. The allowable unit stresses, load and general engineering requirements for both the temporary support and underpinning of structures that are not within the jurisdiction of the Transit Authority, such as other Railroads, highway structures, bridges, piers, etc., must conform to the requirements established by the controlling agency or authority.

LATERAL EARTH PRESSURE DISTRIBUTION

AND

TEMPORARY EARTH RETAINING STRUCTURES

Lateral Earth Pressure Distribution And Temporary Earth Retaining Structures

NOMENCLATURE

b_f	Flanged width of soldier beam (ft.)
C_μ	Undrained Shear Strength of clay = cohesion (psf)
C_a	Surface adhesion of clays (See Reference Standard RS-6, Fig. 13-1) (psf)
D	Required penetration of soldier beam below subgrade (ft.)
D_r	Relative density (%)
d	Depth of soldier beam, property of section (ft.)
F_S	Factor of safety (dimensionless)
G_S	Specific gravity of solids (excludes Air & Water) (dimensionless)
H	Depth of excavation, from ground surface to subgrade which will yield the most critical lateral earth pressure distribution (ft.)
H_C	Thickness of clay layer (ft.)
H_S	Thickness of granular layer (ft.)
h	Depth from ground water table to bottom of excavation (ft.)
h_l	Depth of first level of bracing (ft.)
h_{gwd}	Depth from ground surface to ground water table (ft.)
h_2	Depth from intermediate brace level to subgrade (ft.)
K_a	Coefficient of active earth pressure

$K = \tan^2 (45^o - \phi/2)$ Granular Soil

$K = 1 - m\, 4C_\mu/\gamma H$ Cohesive Soil

(m = 1 except as noted)

K_P	Coefficient of passive earth pressure

K_h Earth pressure coefficient for cohesionless soils on piles; averages 0.5 for $\phi = 30$, 1.0 for $\phi = 45$
(Reference Standard RS-6, pg. 7-13-13)

L Distance between soldier beams (ft.)

m Reduction factor to be applied when N exceeds 6. Less than 1 for metastable (extrasensitive quick) clays

N_S Dimensionless stability number = $\gamma H/C\mu$

N_γ Bearing capacity factor)

N_q Bearing capacity factor) From figure 11-1, pg. 7-11-2

N_c Bearing capacity factor) Reference Standard RS-6

P_{sb} Vertical reaction on soldier beam from wheel loading as positioned on deck beam to produce maximum reaction (kips)

P_{db} Axial load on deck beam to be taken as acting concentric to beam axis (kips)

Pa Resultant active earth pressure (kips or K/ft.)

Pp Resultant passive earth pressure (kips of K/ft.)

Pe Active earth pressure unit stress equivalent for stratified soils (psf)

Pp Unit passive earth pressure (psf)

$Pp = \gamma D K_p + 2C\mu$ $K_p = \gamma D \tan^2 (45^o + \phi/2) + 2C_\mu \tan (45^o + \phi/2)$

This is the general form, which reduces to:
$P_p = \gamma D K_p = \gamma D \tan (45^o + \phi/2)$ Granular Soil ($C_\mu = 0$)

$P_p = \gamma D = 2C_\mu$ Cohesive Soil ($\phi = O$)

Q Maximum vertical reaction on soldier beam from decking system (kips)

R_{db} Design strut load, probably from deck beam (kips)

R_b Design strut load (kips)

R "Fictitious" reaction at subgrade (kips)

W Weight of soldier beam (kips/ft)

w Water content of soil (%)

δ Wall friction angle (degree) See table 10-1, pg. 7-10-7

tan δ Wall friction factor (degree) Reference Standard RS-6

γ/γ_{moist} Unit weight of soil plus the weight of water in the voids (pcf)

γ_b Buoyant unit weight of soil (pcf)

γ_d Dry unit weight of soil (pcf)

γ_w Unit weight of water = 62.4 pcf (65 pcf for sea water)

γ_1 Average unit weight of soil above subgrade (kcf)

γ_2 Average unit weight of soil below subgrade (kcf)

γ_{min} Minimum dry unit weight of soil at 0% relative density (pcf)

γ_{max} Maximum dry unit weight of soil at 100% relative density (pcf)

γ_{sat} Unit weight of soil at 100% Saturation (pcf)

ϕ Angle of internal friction (degrees)

ϕ_1 Angle of internal friction in compacted zone around soldier beam tip. Assumed to be about 5^o larger than ϕ,
$\phi_1 = \phi + 5^o$

GWT Ground Water Table

γ_c Average unit weight of clay (pcf)

γ_s Average unit weight of sand (pcf)

LATERAL EARTH PRESSURE DISTRIBUTION AND TEMPORARY EARTH RETAINING STRUCTURES

GENERAL PROVISIONS

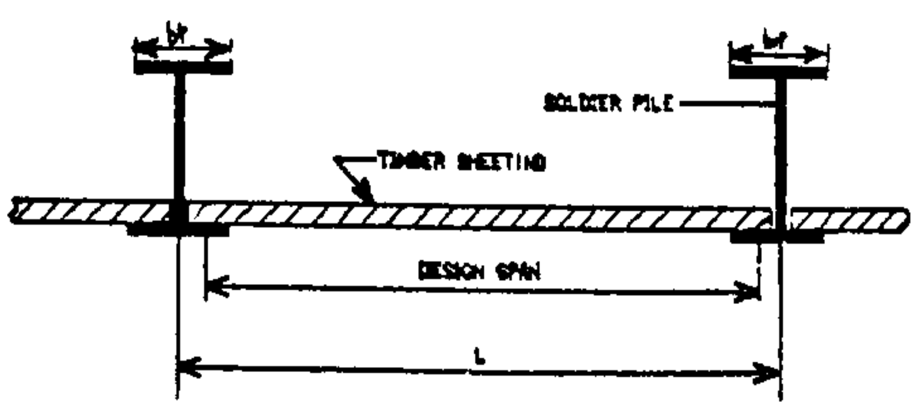
1. Steel soldier beam and horizontal timber sheeting earth retaining structures in soils predominantly composed of sands and gravels are assumed to be in a drained condition where natural seepage through the timber sheeting or mechanical dewatering effectively draws down the original ground water table to the bottom of the excavation.

2. Steel soldier beam and horizontal timber sheeting earth retaining structures in silty and clay soils are assumed to be in an undrained condition. In an undrained condition, the lateral earth pressure distribution is to be based on the assumption that the soil below the original ground water table is fully saturated because of the tendency of these types of soil to retain the water for a long period of time due to their low permeability.

3. In earth retaining structures that will not permit natural seepage in sufficient quantity to lower the ground water table to the bottom of the excavation, such as slurry walls and interlocking steel sheeting, hydrostatic pressure must be included in addition to lateral earth pressure and lateral forces from surface surcharge loads. See sections not dewatered, Pages LP-12 to LP-15.

   The buoyant unit weight is to be used in this case for computation of overburden pressures below the in-situ ground water table.

4. The span to be used for horizontal timber sheeting and horizontal steel plate sheeting shall be the maximum center to center spacing between soldier beams minus one-half the flange width "b_f" of the soldier beams.



5. Soldier beams which are driven from the surface, may be assumed as fully laterally supported in the plane of the timber sheeting if the timber sheeting is placed against the inside flanges of the soldier beams (toward the excavation); otherwise, bracing must be provided. The primary consideration is that there is in-situ soil adjacent to both sides of the soldier beam web and the compression flange due to flexure.

   In the plane perpendicular to the timber sheeting, the effective braced length shall be taken as the distance between braced points.

   Soldier beams which are dropped into augured holes are to be considered laterally unsupported between braced points in that plane.

   Soldier beams may be assumed to be fully braced in both planes below the bottom of the excavation subgrade.

6. The engineering properties of soils for design:

   A) For granular soils, the following may be taken in the absence of more specific data:

      1) γmoist = 115pcf (used above ground water level)

      2) γ_b = 160pcf (used below ground water level)

      3) Value of ϕ: It will usually be satisfactory to use a value of $\phi = 30^o$

   In conjunction with Reference Standard RS-5 and RS-6, an analysis of the boring logs and laboratory tests of the soil may indicate that a larger value than 30^o for the ϕ angle is permissible. However, attention is directed to the fact that granular soils in New York City with values of ϕ less than 30^o do exist, and thus caution should be exercised in the determination of this value.

   B) For Clay - Soils:

   Pertinent design data must be interpreted for each case using analysis of boring logs, appropriate references, and laboratory testing of representative soil samples.

7. To compute the strut loads, assume the development of a hinge in the soldier beam at each strut level. The soldier between each pair of hinges is to be assumed as a simply supported beam. A fictitious strut may be assumed to exist at the bottom of the excavation, providing the penetration required to develop a hinge at the bottom of the excavation is satisfied. This method is applicable for tie-back systems also.

8. The design of struts, walers, and soldier beams or sheet piling must be checked for the several stages of partial excavation and backfilling which loading may be more critical than the loading after completion of the excavation. A lateral support (Struts or Tiebacks) installation and removal schedule shall be submitted to the Engineer for approval.

9. Rock Tie-Backs will be permitted by the Engineer within the following limitations:

   a. A testing monitoring, reloading and unloading procedure for the tie-back system shall be submitted to the Engineer for approval.

   b. Design stresses shall not exceed 60% of the ultimate tensile strength of the ties.

   c. The vertical and horizontal components of the tie-back load shall be considered in the determination of the penetration, bearing, and/or stability requirements of the temporary earth retaining structure of which the tie-backs are a component.

   d. Tie-backs will not be permitted to extend into private property or within the vicinity of an existing foundation that may be adversely affected by the disturbance of the soil in its proximity.

   e. All tie-backs shall be released by the completion of construction and all loads transferred to the permanent structure uniformly.

10. Earth Anchor Tie-backs will be permitted by the Engineer only when existing conditions preclude the use of more conventional systems, such as struts or rock tie-backs. Once their applicability is established, earth anchors will be permitted under the limitations of paragraph 9 page LP-7 with the additional requirements:

    a. Earth anchors will not be permitted in soils that may exhibit creep characteristics.

b. The effective design length of an earth anchor tie-back is the portion of the earth anchor tie-back that projects beyond the influence of the "slip plane". The slip plane is defined as a line with its origin at the base of the excavation (at the intersection of the subgrade with the sheeting line) and projecting to the ground surface. The appropriate slip plane must be determined from in-situ soil conditions. Under no circumstances will the slip plane be less than 30^{o} with the vertical face of the excavation.

11. The required length of soldier beam embedment below the bottom of the excavation to develop a hinge and/or to provide resistance to axial loads against bearing capacity failure of the soil shall be determined by the formulas given on pages LP-19 to LP-22. Axial load requirements must be confirmed to the Engineer on the basis of dynamic pile drive resistance formulas, load tests or recognized analyses of local soil conditions.

Note: Dynamic pile driving resistance formulas should not be relied upon to determine the axial load capacities of piles in fine - grained soils, i.e., silts and clays.

12. For design of interlocking steel sheeting, see Reference Standard RS-8.

13. The stability of the base of the excavation must also be examined. For this and other considerations not previously indicated, such as piping due to seepage, unbalanced external forces, etc., see Reference Standard RS-6 and Reference Standard RS-7.

14. Design of Lateral Earth support systems for test pits and underpinning pits shall use loading criteria LP-11 to LP-18.

15. As a minimum, struts to be "welded tight" against soldier beam.

16. If design criteria and/or methods given in any of the Reference Standards differs with a criteria and/or method explicitly stated in the Field Design Standards, the Field Design Standard criteria and/or method will govern.

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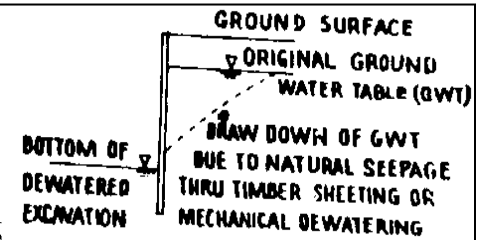
PRESSURE DISTRIBUTION

FOR

TEMPORARY EARTH RETAINING STRUCTURES

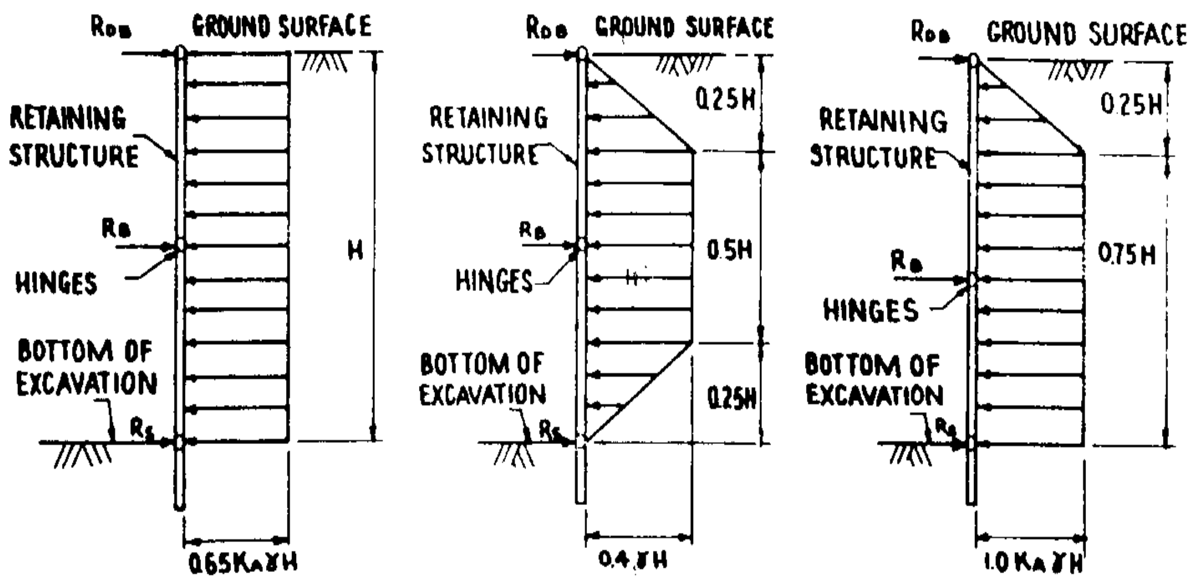
DEWATERED SECTIONS

LATERAL EARTH PRESSURE DISTRIBUTION
FOR
TEMPORARY EARTH RETAINING STRUCTURES



DEWATERED SECTIONS *(GWT ASSUMED BELOW BOTTOM OF EXCAVATION

GRANULAR SOIL | STIFF-FISSURED CLAYS (Ns≤6) | SOFT TO MEDIUM CLAYS (Ns>6)

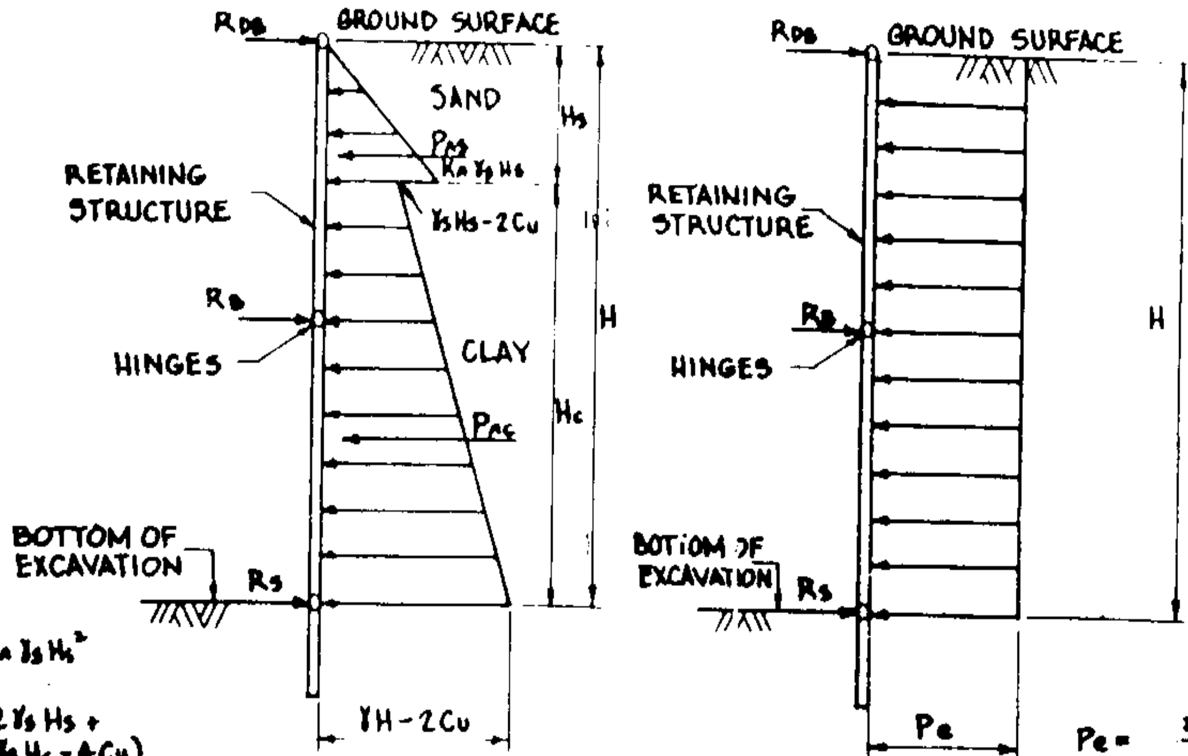


Note: Sand is assumed drained (See note 1 of general provisions)
Clay is assumed undrained (See note 2 of general provisions)
Pressure envelopes are semi-amprical apparent maximum pressure diagrams. See reference standard RS-7.

LATERAL EARTH PRESSURE DISTRIBUTION FOR TEMPORARY EARTH RETAINING STRUCTURES

DEWATERED SECTIONS (GWT ASSUMED TO BE BELOW BOTTOM OF EXCAVATION)

STRATIFIED SOIL



$P_{AS} = \frac{1}{2} K_A \gamma_S H_S^2$

$P_{AC} = \frac{H_S}{2}(2\gamma_S H_S + \gamma_C H_C - 4C_U)$

$\gamma H = \gamma_S H_S + \gamma_C H_C$

$P_e = \frac{\Sigma P_{AS} + \Sigma P_{AC}}{H}$ (PSF)

ORIGINAL PRESSURE DISTRIBUTION

REDISTRIBUTION

PRESSURE DISTRIBUTION

FOR

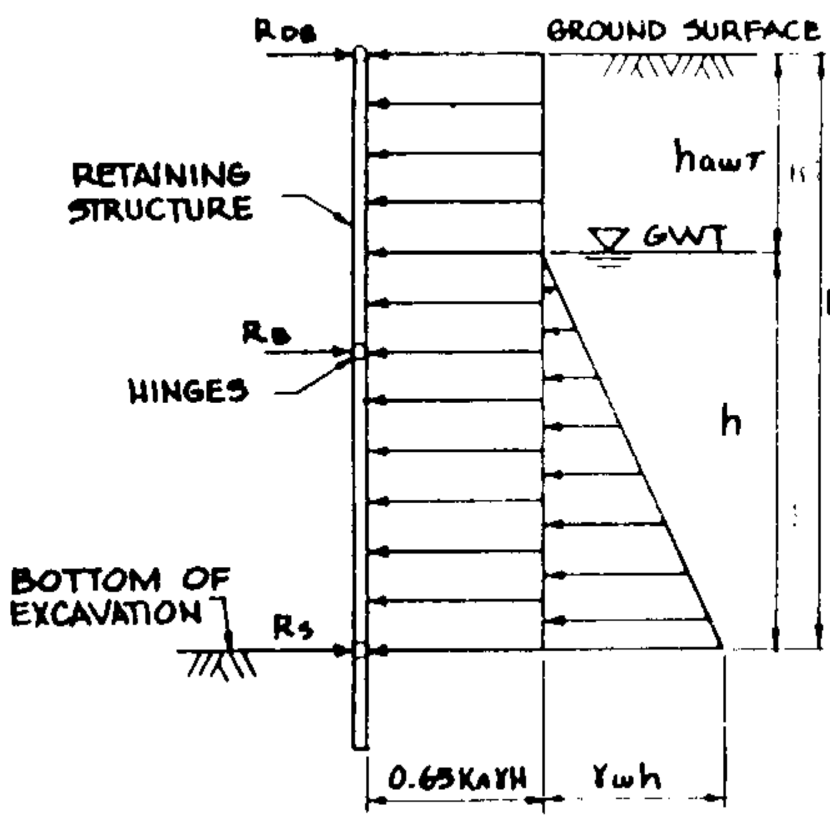
TEMPORARY EARTH RETAINING STRUCTURES

SECTION NOT DEWATERED

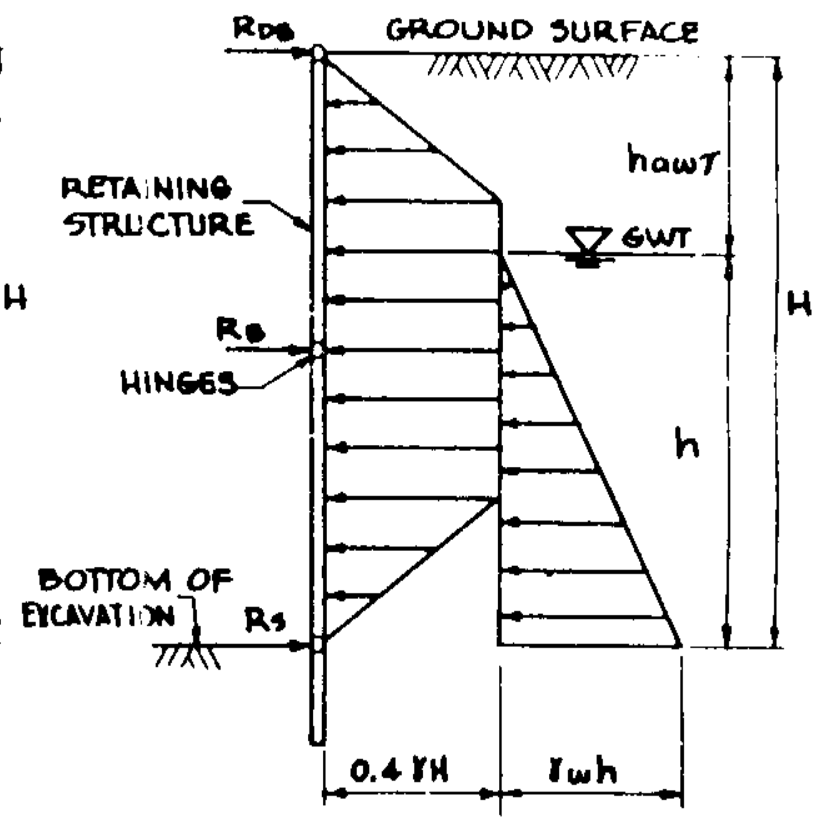
LATERAL EARTH PRESSURE DISTRIBUTION
FOR
TEMPORARY EARTH RETAINING STRUCTURES

SECTIONS NOT DEWATERED (SEE NOTE 3 GENERAL PROVISIONS)

GRANULAR SOIL



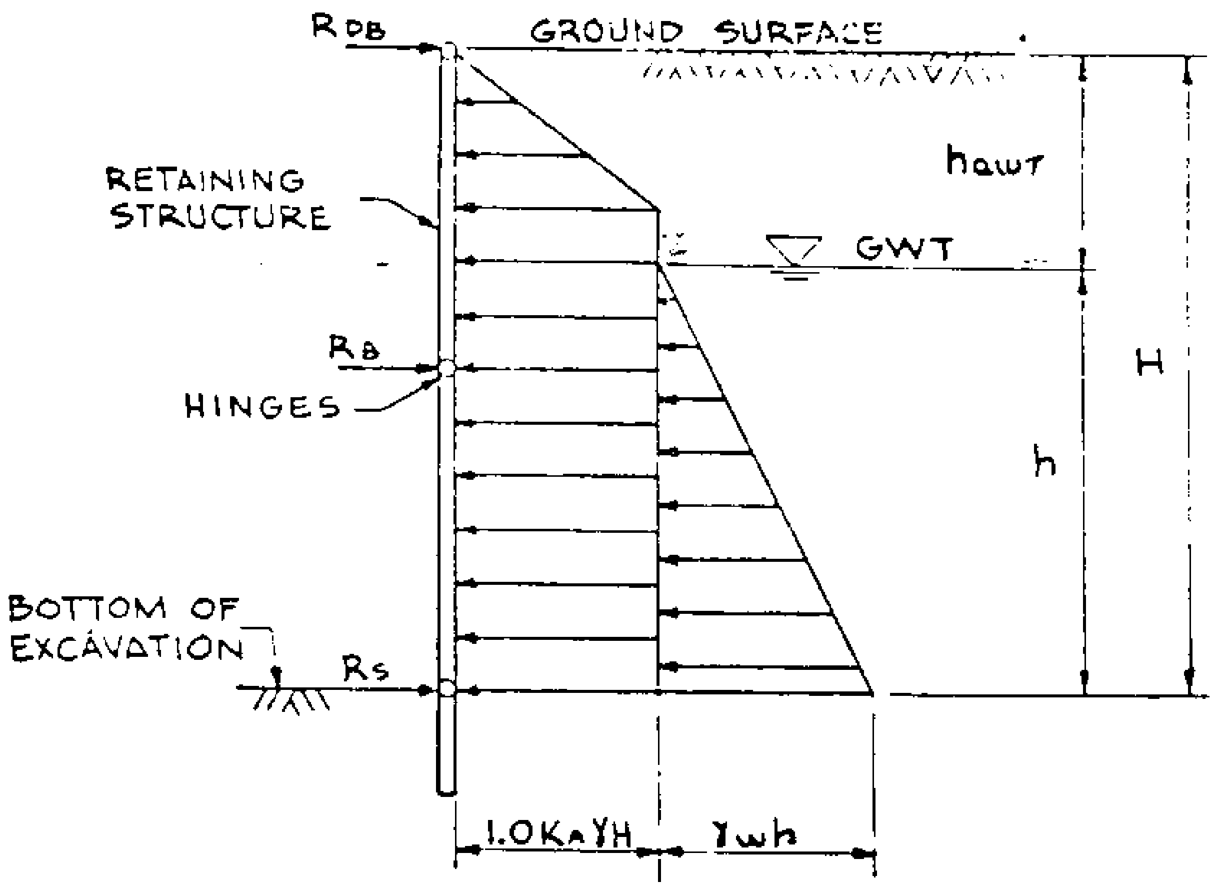
STIFF-FISSUED CLAYS (Ns ≤ 6)



LATERAL EARTH PRESSURE DISTRIBUTION FOR TEMPORARY EARTH RETAINING STRUCTURES

SECTIONS NOT DEWATERED (SEE NOTE 3 OF GENERAL PROVISIONS)

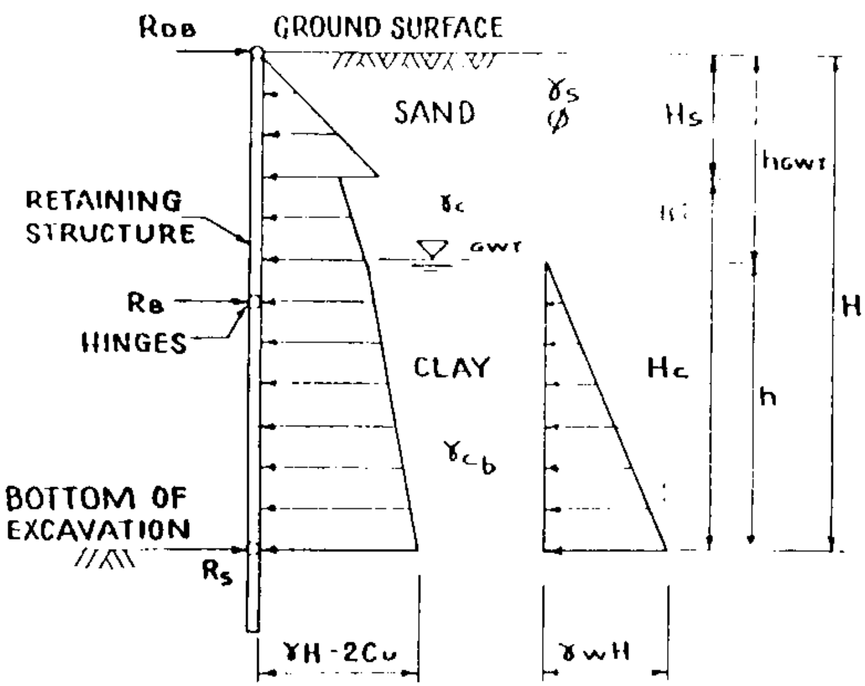
SOFT TO MEDIUM CLAYS (Ns > 6)



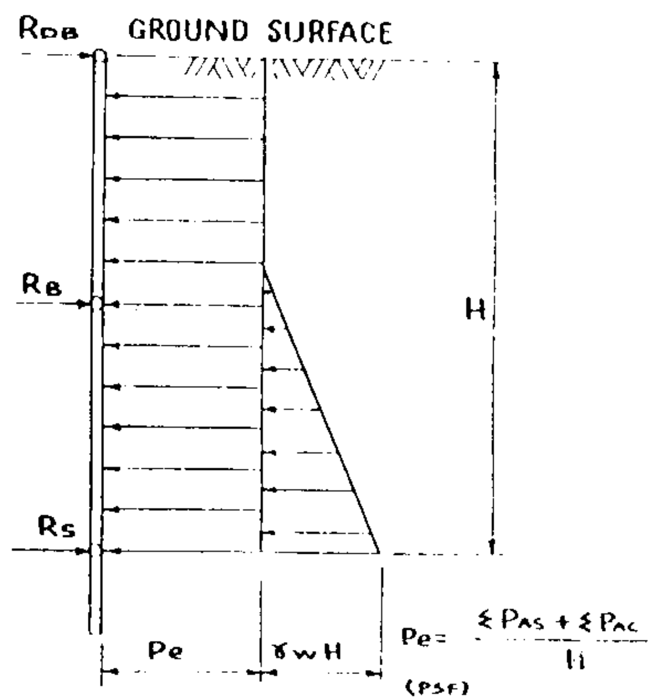
LATERAL EARTH PRESSURE DISTRIBUTION
FOR
TEMPORARY EARTH RETAINING STRUCTURES

SECTIONS NOT DEWATERED

STRATIFIED SOIL



ORIGINAL PRESSURE DISTRIBUTION
(see page LP-17)



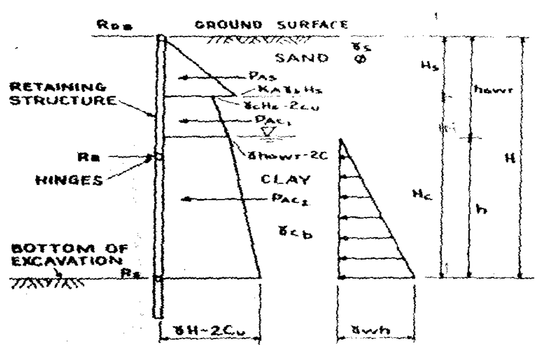
REDISTRIBUTION

LATERAL EARTH PRESSURE DISTRIBUTION

FOR

TEMPORARY EARTH RETAINING STRUCTURES

SECTIONS NOT DEWATERED
STRATIFIED SOIL



$$\gamma H = \gamma_S H_S + \gamma_C (h_{GWT} - H_S) + \gamma_{c_b} (h)$$

$$P_{A_S} = 1/2\ K_A \gamma_S H_S^2$$

$$P_{AC_1} = \frac{(h_{GWT} - H_S)}{2}\left[(\gamma_S H_S - 2Cu) + (\gamma h_{GWT} - 2Cu)\right]$$

$$P_{AC_2} = \frac{h}{2}\left[(\gamma h_{GWT} - 2Cu) + (\gamma H - 2Cu)\right]$$

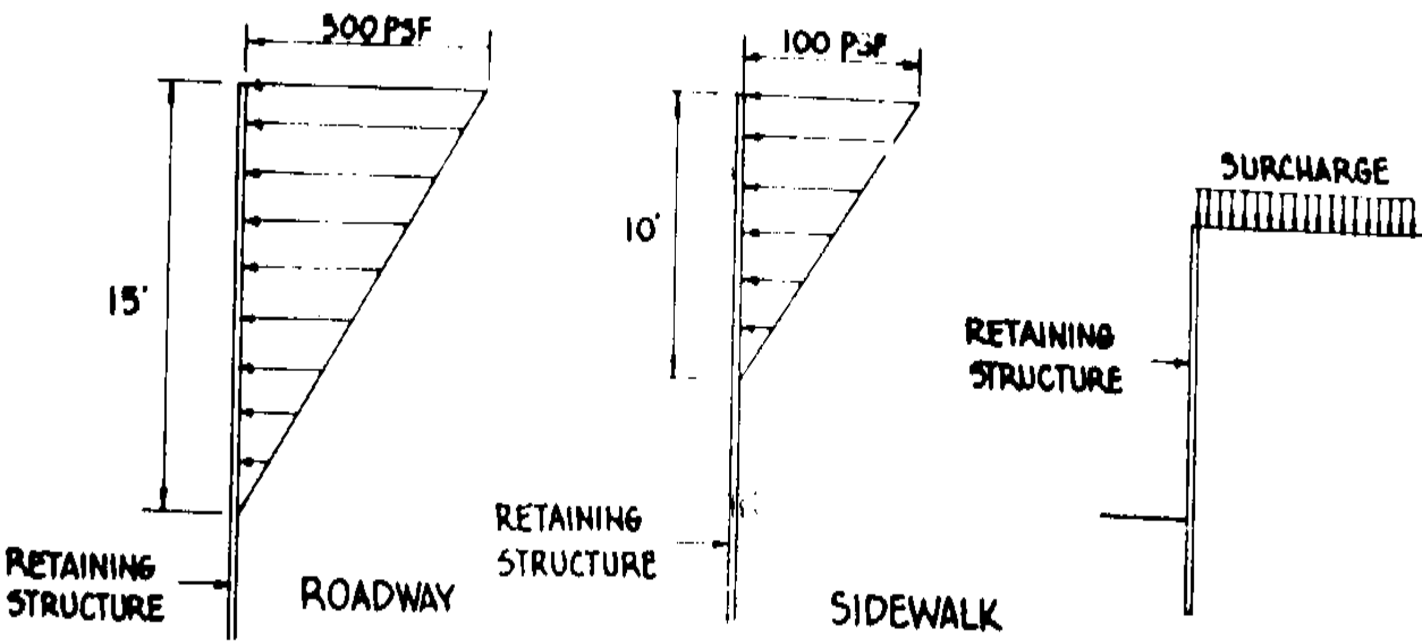
$$\sum P_{AC} = P_{AC_1} + P_{AC_2}$$

$$P_A = \frac{\sum P_{AS} + \sum P_{AC}}{H} \quad \text{(psf)}$$

ORIGINAL PRESSURE DISTRIBUTION

LATERAL PRESSURE DUE TO SURCHARGE
FOR
TEMPORARY EARTH RETAINING STRUCTURES

FOR SECTIONS DEWATERED AND NOT DEWATERED



Note: Roadway lateral pressure is based on an assumed construction equipment roadway loading of 600 psf. This surcharge should be applied unless it can be shown that field conditions do not allow major construction equipment to be positioned and supported on the sidewalk or roadway adjacent to the retaining structure. In all other conditions a sidewalk surcharge loading of 200 psf should be applied. The lateral pressure due to surcharge shown applies to Flexible retaining structures. For concrete-slurry wall structures the lateral pressures indicated above must be increased by a factor of 2.

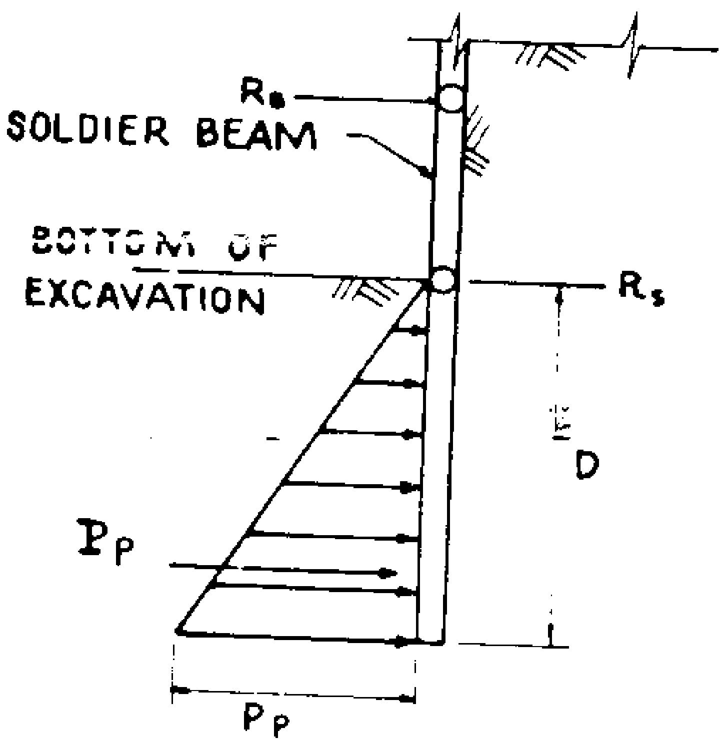
PENETRATION OF SOLDIER BEAMS*

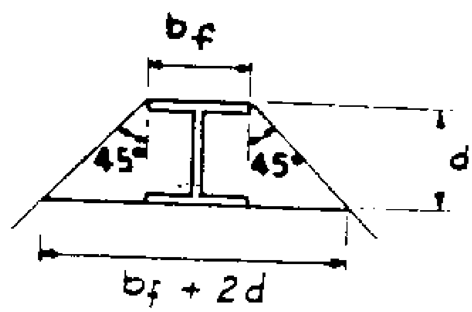
USE LARGER VALUE OF PENETRATION AS OBTAINED FROM CASE I, IIA OR IIB

CASE I - For All Types of Soil:
Penetration needed to develop "HINGE" at fictitious support, R_s

CRITERIA: THE PENETRATION, D, IS THE DEPTH REQUIRED TO DEVELOP THE NECESSARY PASSIVE PRESSURE TO RESIST THE FORCE AT THE FICTITIOUS SUPPORT, R_s

$\therefore R_s = P_p$





$$D = 2Rs/Pp\ (b_f + 2d)$$

Minimum penetration required to develop "HINGE" is 6'0"
This case is only required if strut and soldier beam design is based in part on the development of a hinge at bottom of excavation

* For penetration of interlocking steel sheeting see reference standard Rs-8.

PENETRATION OF SOLDIER BEAMS

USE LARGER VALUE OF PENETRATION AS OBTAINED FROM CASE I, IF APPLICABLE, OR CASE IIA

CASE IIA - For Granular Soils

Penetration needed to develop resistance to axial load against bearing failure

THE FOLLOWING FORMULAR IS FOR A GRANULAR SOIL BASED ON REFERENCE STANDARD RS-6. CHAPTERS 12 & 13 FOR DEEP FOUNDATION AND PILE FOUNDATION STATIC ANALYSIS MODIFIED TO ACCOUNT FOR EXCAVATION AND INSTALLATION OF HORIZONTAL TIMBER SHEETING.

1.1
$$\left[\frac{Q+W(H+D)}{b_f d}\right]=\frac{.65K_A\tan\sigma}{d}\left[\gamma_1 H^2+\gamma^2 HD+\gamma_1 D^2\right]+$$
$$\frac{\gamma 2D2\tan\sigma}{b_f d}\left[.5Kpbf+K_H d\right]+\gamma_2 DNq+.4\gamma_2 dN\gamma+.1\gamma_2 D$$

NOTE THE WALL FRICTION FACTOR, TAN σ = .3 (RS-6. TABLE 10-1, PG 7-10-7). TO DETERMINE BEARING CAPACITY FACTORS, Nq AND Nγ, ANGLE OF INTERNAL FRICTION IN COMPACTED ZONE AROUND SOLDIER TIP 5⌖LARGER THAN ∅.∅.=∅ + 5°

THE ABOVE EQUATION IS OF THE FORM AD^2 + BD + C = O WHERE

$$A = \gamma_2\left[.195K_A+.15Kp+3d\,K_H/b_f\right]$$

$$B = (.195KAH/d)(\gamma_1+\gamma_2)+\gamma_2 Nq-1.1W/b_f d+.1\gamma_2$$

$$C = .195\,K_A\gamma_1 H^2/d+4\gamma_2 dN\gamma-1.1/b_f d\left[Q+WH\right]$$

THIS FORMULA IS BASED ON A STATIC ANALYSIS AND IS TO BE USED IN THE PREPARATION OF SHOP DRAWINGS. SEE NOTE II OF GENERAL PROVISIONS.

Penetration of Soldier Beams (continued)

IN ACCORDANCE WITH NOTE 6A OF THE GENERAL PROVISIONS THIS FORMULA RESULTS IN THE FOLLOWING EXPRESSION FOR THE SPECIFIC SOIL PARAMETERS GIVEN. IF THE SOIL PARAMETERS FOR THE GIVEN SITE DIFFER FROM THOSE IN NOTE 6A THE FORMULA ON THE PRECEDING PAGE IS TO BE USED.

$$\gamma_1 = 0.115\text{KCF}$$

$$\gamma_2 = 0.060\text{KCF}$$

$$\phi = 30^\circ \begin{cases} K_A = 1/3 \\ K_P = 3 \\ K_H = 1/2 \end{cases} \qquad \phi_1 = 35 \begin{cases} N\gamma = 20 \\ Nq = 36 \end{cases} \qquad \begin{matrix} \text{RS-6 Fig. 11-1} \\ \text{pg. 7-11-2} \end{matrix}$$

$$A = .0309/d + .009/b_f$$

$$B = .01136\ H/d + 2.166 - 1.1\ w/b_f d$$

$$C = .0075\ H^2/d + .48d - \frac{1.1}{b_f d}\left[Q + WH\right]$$

$$AD^2 + BD + C = 0$$

$$D = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A}\ (\text{FT.})$$

PENETRATION OF SOLDIER BEAMS

USE LARGER VALUE OF PENETRATION AS OBTAINED FROM CASE I, IF APPLICABLE, OR CASE IIB

CASE IIB - For Cohesive Soil:
Penetration needed to develop resistance to axial load against bearing failure

THE FOLLOWING FORMULA IS FOR A COHESIVE SOIL BASED ON REFERENCE STANDARD RS-6 CHAPTER 13 FOR PILE FOUNDATION STATIC ANALYSIS MODIFIED TO ACCOUNT FOR EXCAVATION AND INSTALLATION OF HORIZONTAL TIMBER SHEETING**.**

$$\boxed{D = \frac{1.1(Q + WH) - b_f\, d\ CuNc - C_A\ Hb^2}{2C_A(b_f\ + d) + .1\gamma\, b_f\, d - 1.1w}}$$

SEE NOT II OF GENERAL PROVISIONS

ENR. PILE FORMULA

$$B/INCH = \left(\frac{1}{2E / P - 0.1}\right) * 1.25$$

$$or$$

$$P = \frac{B/INCH\left(2E - 0.1P\right)}{1.25}$$

*** FACTOR OF SAFETY**

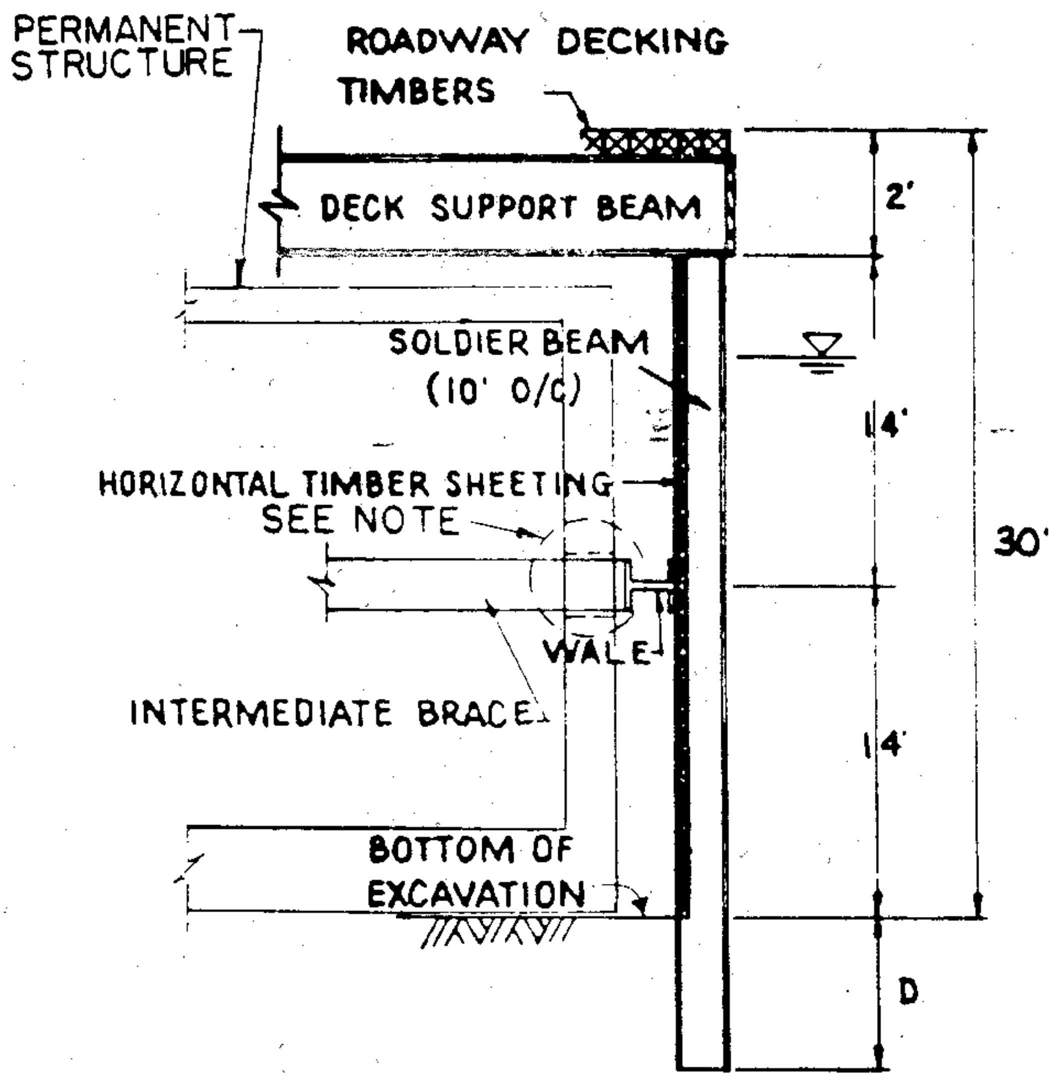
LATERAL EARTH PRESSURE DISTRIBUTION

FOR

TEMPORARY EARTH RETAINING STRUCTURES

DESIGN EXAMPLE I - GRANULAR SOIL

DESIGN EXAMPLE I



NOTE:

USE OF WINDOW FOR STRUTS MUST BE APPROVED BY ENGINEER

DESIGN EXAMPLE I

GIVEN - THE SOLDIER BEAM AND HORIZONTAL TIMBER SHEETING TEMPORARY EARTH RETAINING STRUCTURE SHOWN ON PREVIOUS PAGE

- THE EXISTING CONDITIONS AS INTERPRETED FROM BORING LOG DATA:

1) GWT 9 FEET BELOW GROUND SURFACE
(NOTE: AS PER NOTE 1 OF GENERAL PROVISIONS, EFFECTS OF THE GROUND WATER TABLE WILL BE NEGLECTED BECAUSE WATER CAN ENTER EXCAVATION THROUGH HORIZONTAL TIMBER SHEETING)

2) SOIL: GRANULAR SOIL →Br. c-f SAND, trace Silt
3) RELATIVE DENSITY: N_{avg} = 20 BLOWS/6"
(2" O.D. - 1-3/8" I.D. SPLIT SPOON SAMPLER, 140 lb HAMMER, 30" DROP)

- WHEEL LOADS WERE POSITIONED TO PRODUCE MAXIMUM DECK BEAM REACTION ON SOLDIER BEAM OF 50^k. (50^k REACTION ASSUMED FOR DESIGN EXAMPLE I ONLY. SEE DESIGN EXAMPLE III FOR DESIGN OF DECKING BEAM AND EVALUATION OF MAXIMUM REACTION)

DETERMINE:
- I. SOIL DESIGN PARAMETERS
- II. LATERAL PRESSURES
- III. DESIGN LOADS

DESIGN:
- IV. SOLDIER BEAM
- V. WALE
- VI. INTERMEDIATE BRACE
- VII. HORIZONTAL TIMBER SHEETING

DESIGN EXAMPLE I (CON'T)

1. SOIL DESIGN PARAMETERS

ENGINEERING PROPERTIES OF IN-SITU SOIL FOR DESIGN TO BE DETERMINED USING REFERENCE STANDARD RS-5 AS PER NOTE 8 OF GENERAL PROVISIONS.

BASED ON BLOWCOUNT DATA THE COMPACTNESS RATING OF THE SOIL IS MEDIUM COMPACT (+) WHICH CORRESPONDS TO AN IN-PLACE RELATIVE DENSITY OF APPROXIMATELY 65% (REFERENCE STANDARD RS-5, FIG 6)

FOR C - f SAND, trace silt

$$\left.\begin{aligned}\gamma_{max} &= 111\text{PCF}\\ \gamma_{min} &= 92\text{ PCF}\end{aligned}\right\}\text{RS - 5, TABLE II AND FIGS. 3 AND 4}$$

$$\phi = 37^\circ \qquad \text{RS - 5, FIG. 14}$$

$$D_R = \frac{1/\gamma_{min} - 1/\gamma}{1/\gamma_{min} - 1/\gamma_{max}}(100)$$

$$.65 = \frac{1/92 - 1/\gamma}{1/92 - 1/111} = \frac{.0109 - 1/\gamma}{.0109 - .0090}$$

$$1/\gamma = .0097 \qquad \therefore \gamma = 103\text{PCF (DRY)}$$

ASSUME W = 10% (WATER CONTENT)

$$\gamma_T = (1 + W)\gamma d$$

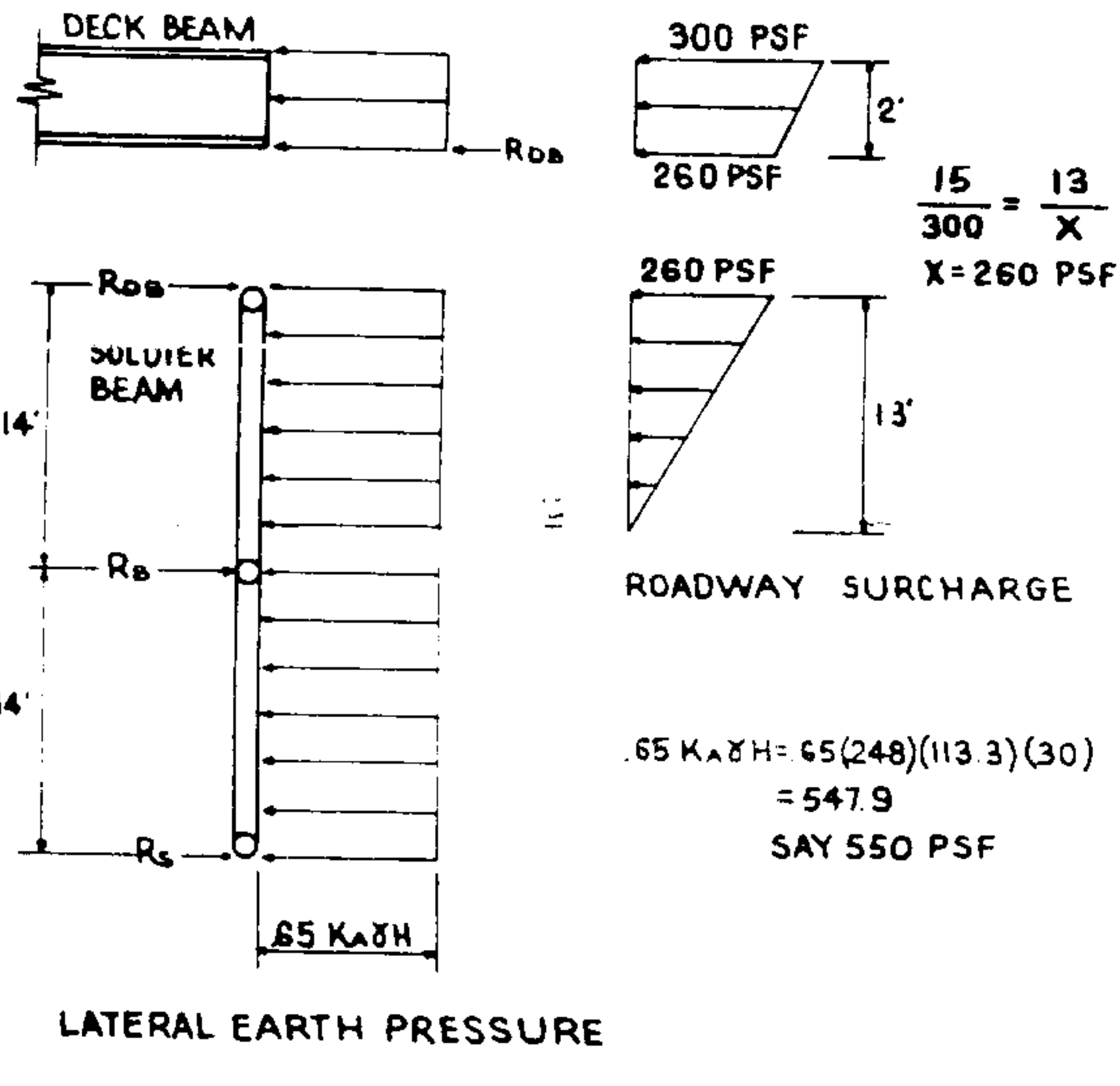
$$= (1.1)(103) = \underline{113.3\text{PCF} \qquad \gamma_T}$$

$$K_A = \frac{1 - \text{SIN } \phi}{1 + \text{SIN } \phi}$$

$$= \frac{1 - .602}{1 + .602} = \underline{.248 \qquad K_A}$$

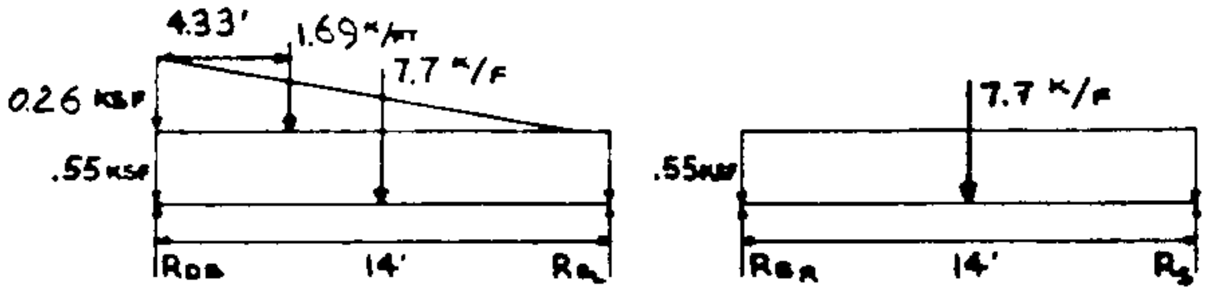
DESIGN EXAMPLE I (con't)

II. LATERAL PRESSURES



DESIGN EXAMPLE I (CON'T)

III. DESIGN LOADS



SEE NOTE 7 OF GENERAL PROVISIONS

$$R_{DB} = 7/14(7.7) = 3.85$$
$$9.67/14(1.69) = \underline{1.17}$$
$$5.02 \text{ K/FT.} \uparrow \qquad R_{DB}$$

$$R_{B_L} = 7/14(7.7) = 3.85$$
$$4.33/14(1.69) = \underline{.52}$$
$$4.37 \text{ K/FT.}$$

$$R_{B_R} = 7/14\ (7.7) = 3.85 \text{ K/FT.}$$
$$R_B = R_{B_L} + R_{B_R} = 4.37 + 3.85 = 8.22 \text{ K/FT.} \uparrow \qquad R_B$$
$$R_S = 7/14(7.7) = 3.85 \text{ K/FT.} \uparrow \qquad R_S$$

LOCATE POINT OF ZERO SHEAR

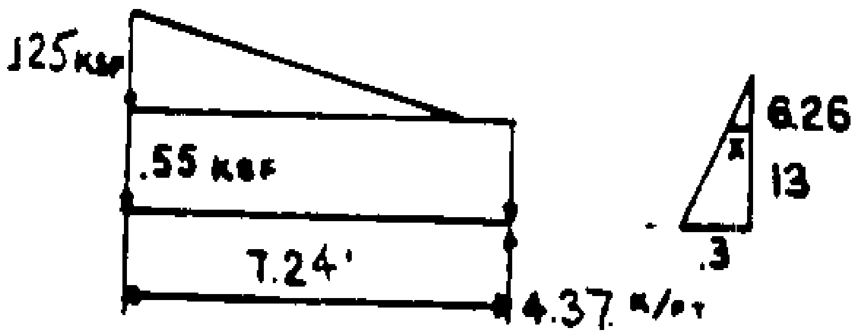
$$R_{B_L} - .55\,x - \frac{0.26(x-1)}{1.3}\ \frac{(x-1)}{2} = 0$$
$$4.37 - .55x - .010\left(x^2 - 2x + 1\right) = 0$$
$$.\qquad 010x^2 + .53x - 4.36 = 0$$
$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$x = \frac{-.53 \pm \sqrt{.281 + 4(.010)(4.36)}}{2(.010)} = \frac{-.53 \pm .675}{.02}$$
$$x = 7.24 \text{FT FROM } R_{B_L}$$

DESIGN EXAMPLE I (CON'T)

III DESIGN LOADS (CON'T)

COMPUTE MAXIMUM MOMENT



$$\frac{X}{83.24} = \frac{.26}{13}$$
$$XX = .125KSF$$

$$M = 4.37\,(7.24) - .55(7.24)\frac{7.24}{2} - 6.24\left(\frac{.125}{2}\right)\left(\frac{6.24}{3}\right)$$

$$M = 31.639 - 14.415 - 0.811 = 16.41\ \frac{\text{K-FT}}{\text{FT}}$$

FOR SOLDIER BEAMS SPACED 10' O / C

$M_{MAX} = 16.41 \cdot (10) = \underline{164.1\text{K - FT} \qquad M_{MAX}}$

FROM DECK BEAM CALCULATIONS

WHEEL LOADINGS WERE POSITIONED TO PRODUCE MAXIMUM REACTION ON SOLDIER BEAM. $P_{SB} = 50^K$ (ASSUMED FOR THIS DESIGN EXAMPLE)

$\underline{50^K \qquad P_{SB}}$

AXIAL LOAD ON DECK BEAM

$$P_{DB} = R_{DB} + \text{EARTH} + \text{SURCHARGE}$$

$$P_{DB} = 5.02 + .55(2) + \frac{2}{2}(.3+.26) = 6.38\,\text{K/FT.}$$

FOR SOLDIER BEAMS SPACED 10' O/C

$P_{DB}\ 6.38\ \text{K/FT.} \times 10' = \underline{63.8^K \qquad P_{DB}}$

DESIGN EXAMPLE I (CON'T)

IV DESIGN OF SOLDIER BEAM

$P = 50^K$ NOTE: SOLDIER BEAMS GENERALLY
$M = 168.7$ K - FT LIMITED TO W12 OR W14 SECTIONS.
$L = 14$FT REFERENCE: AISC 7TH EDITION

TRY W12 x 65 (NON-COMPACT SECTION IN A36)

$$\left.\begin{array}{l} A = 19.1 \text{ in}^2 \\ S_x = 88.0 \text{ in}^3 \\ r_x = 5.28 \text{ in} \\ r_y = 3.02 \text{ in} \end{array}\right\} \text{pg 1-38}$$

$f_a = P/A = 50/19.1 = 2.62$KSI
$f_b = M/S = 168.7(12)/88 = 23.0$KSI
$F_b = 23.4$ KSI (pg 2-43)

HOWEVER FOR TEMPORARY STRUCTURES $F_b = 1.2\ F_b$ (AISC)
$\therefore F_b = 12(23.4) = 28.1$KSI
$Kl/r = (14)(12)/5.28 = 31.8$ (SEE NOTE 5 OF GENERAL PROVISIONS)
$\therefore F_a = 19.81$KSI (TABLE 1-36, pg 5-84)
$f_a/F_a = 2.62/19.81 = 0.132 < .15$ $\therefore$ USE AISC FORMULA (1.6-2)
$f_a/F_a + f_b/F_b \le 1$
$.132 + 23/28.1 = .132 + .819 = .951 < 1$ OK

$\therefore$ USE W12 x65 OR EQUIVALENT FOR SOLDIER BEAMS

PENETRATION OF SOLDIER BEAMS
(SEE NOTE 11 OF GENERAL PROVISIONS)
CASE I PENETRATION REQUIRED TO DEVELOP "PIN"

ASSUME $G_s = 2.67$ $\gamma_d = \dfrac{G_s}{1+e}\gamma_w$

$103 + 103e = 166.6$ $e = 63.6/103 = .617$

$$\gamma_{SAT} = \frac{G_s + e}{1+e}\gamma_w = 126.8\text{PCF}$$

$$\gamma_b = \gamma_{SAT} - \gamma_w = 64.4\text{PCF}$$

DESIGN EXAMPLE I (Con't)

IV. DESIGN OF SOLDIER BEAM (Con't)

$K_P = 1/K_A = 1/.248 = 4.03$
$b_f = 12"$

$D = 2/3 \ R_S/Ppb_f$
$D = 2/3 \ (38.5/.0644(D)\ (4.03)\ (12/12)$
$D = \ 98.9/D \quad \therefore D^2 = 98.9$
$D = 9.94$ FEET SAY 10 FEET

$\therefore$ PENETRATION REQUIRED TO DEVELOPE "PIN" IS 10'

CASE II$_A$: RESISTANCE TO AXIAL LOAD AGAINST BEARING FAILURE

$\gamma_2 = .0644$ KCF

$\left.\begin{array}{l} b_f = 12" \\ d = 12.12" \end{array}\right\} b_f d = 1FT^2$

$H = 28'$

$\gamma_1 = .1133$ KCF
$W = .065 \ K/FT$
$Q = 50K$

$\phi = 37°$ $\qquad \phi_1 = \phi + 5 = 42°$

$K_A = .248$

$K_P = 4.03$

$K_H \left\{\begin{array}{ll} 45° & 1 \\ 37° & K_H \\ 30° & .5 \end{array}\right\} K_H = .733$

$\left\{\begin{array}{l} N\gamma = 73 \\ Nq = 92 \end{array}\right.$ DM - 7 Figure 11 - 1 pg. 7 - 11 - 2

$A = .0644[.195(.248)+.15(4.03)+.3(.733)] = .0562$
$B = [.195(248)(28)]\ (.1777)+.0644\ (92) - 1.1\ (.085) + 0.1\ (0.0644)] = 6.10$
$C = .195(.248)(.1133)(784)+.4(.0644)(73) - 1.1[50 + 1.82] = -50.826$

$.0562 \ D^2 + 6.10 \ D - 50.826 = 0$

$$D = \frac{-6.10 \pm \sqrt{37.21 + 4(.0562)\ (50.826)}}{2(.0562)} = \frac{6.10 \pm 6.974}{.1124}$$

$D = .874/.1124 = 7.78$ FT. SAY 8FT.

DESIGN EXAMPLE I (CON'T)

IV. DESIGN OF SOLDIER BEAM (CON'T)

NOTE: THIS REQUIRED PENETRATION OF 8 FEET, BASED ON A STATIC ANALYSIS, IS TO BE USED FOR ESTIMATING PURPOSES ONLY. THE ACTUAL PENETRATION REQUIRED TO DEVELOP THE DESIGN AXIAL LOAD CAPACITY OF THE SOLDIER BEAM MUST BE VERIFIED AND/OR MODIFIED IN THE FIELD USING STANDARD DYNAMIC PILE DRIVING RESISTANCE FORMULAS, SUCH AS THE ENGINEERING NEWS FORMULA.

∴ REQUIRED PENETRATION OF SOLDIER BEAM
D IS 10 FEET BELOW SUBGRADE
(SEE NOTE II OF GENERAL PROVISIONS)

STABILITY AT BOTTOM OF EXCAVATION

FROM REFERENCE STANDARD RS-6 FIGURE 10-17,
STABILITY AT BASE OF BRACED CUT, pg 7-10-23.

$F_S = 2N_{\gamma 2}\ (\gamma_2/\gamma_1)\, K_A \tan\phi$

FOR GROUND WATER STATIC AT BASE OF CUT:

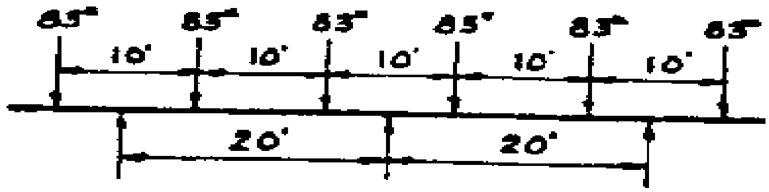
γ_1 = MOIST WEIGHT = 113.3pcf
γ_2 = SUBMERGED WEIGHT = 64.4 pcf
$\phi = 37°$ ∴ $N\gamma = 38$ (Fig 11-i, pg. 7-11-2)
$\tan\phi = .754$

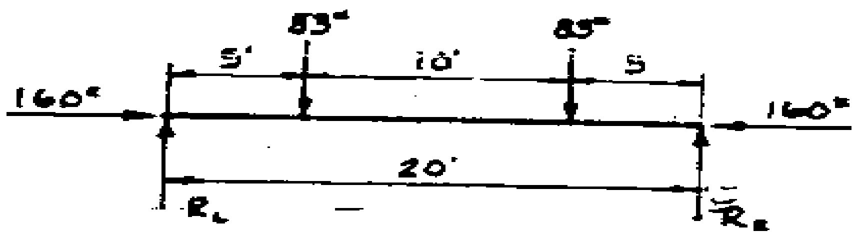
$F_S = 2(38)\ (64.4/113.3)\ (.248)\ (.754)$
$F_S = 8.08 > 1.5$ O.K.

DESIGN EXAMPLE I (CON'T)

<u>V DESIGN OF WALE</u>

ASSUME TRUST FROM PERPENDICULAR WALE
AT CORNER = 160*
ASSUME INTERMEDIATE BRACES EVERY 20 FT.





$R_L = R_E = 15/20\,(83)$

$\underline{\quad 5/20\,(83)}$

$83^{*}\uparrow$

$M_{MAX} = 83(5) = 415_{K-FT}\ (x\text{-AXIS})$

$P = 160^{K}$

$My = wL^2/8 = w(100)/8 = 12.5w_{\ K-FT}$

$\underline{\text{TRY W30 x 99}} \left(\begin{array}{llll} A = 29.1 & S_x = 270 & r_x = 11.7 & \text{pg. } 1\text{-}28 \\ & S_y = 24.5 & r_y = 2.10 & \end{array} \right.$

$$f_a = \frac{P}{A} = \frac{160}{29.1} = 5.50_{KSI}$$

$$f_{bx} = \frac{M_x}{S_x} = \frac{415(12)}{270} = 18.44_{KSI}$$

$$f_{by} = \frac{12.5(.099)\ (12)}{24.5} = 0.606_{KSI}\ (\text{SMALL} \therefore \text{CAN BE NEGLECTED})$$

$Lc = 10.9'\,(\text{pg.}2\text{-}31) > 10'$ $\quad * \therefore$ ALLOWABLE BENDING STRESS, $F_b = 24$ KSI
HOWEVER FOR TEMPORARY STRUCTURES, $F_b = 1.2(F_b) = 28.8$KSI.

* NOTE: ASSUME EACH SOLDIER WELDED TO WALE THEREFORE COMPRESSION FLANGE OF WALE BRACED EVERY 10F

DESIGN EXAMPLE I (Con't

$$\frac{KL}{r_y} = \frac{10(12)}{2.10} = 57.1 \quad \therefore F_a = 17.70_{KSI} \text{ (TABLE 1-36, pg. 5-84)}$$

$$\frac{f_a}{F_a} = \frac{5.50}{17.70} = 0.311 > .15 \therefore \text{USE AISC FORMULA (1.6-1a)}$$

$$\frac{f_a}{F_a} + \frac{C_m f_{bx}}{(1 - f_a / F'_{ex}) F_{bx}} \leq 1$$

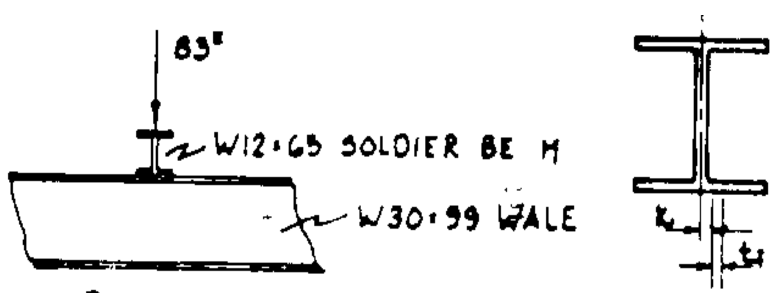
$$C_m = 1.0$$

$$\frac{KL_b}{r_b} = \frac{10(12)}{11.7} = 10.3 \therefore F'_e = 1408_{KSI} \text{ (TABLE 2, pg. 5-94)}$$

$$.311 + \frac{1.0\ (18.44)}{(1 - 5.50/1408)28.8} = .311 + .643 = .954 < 1 \quad \underline{\text{ok}}$$

$\underline{\therefore \text{ USE W30 x 99 OR EQUIVALENT FOR WALE}}$

<u>CHECK WEB CRIPPLING OF WALE</u>



$$N = 2(K_1 + \phi_4)$$
$$N = 2(.813 + .606)$$
$$N = 2.838''$$

$$\frac{R}{t(N + 2_K)} \leq .75 F_y (1.2)*$$

$$\frac{83}{.522\ (2.838 + 2[1.4375])} = \frac{83}{2.98} = 27.9_{KSI} < 32.4_{KSI} \quad \underline{\text{ok}}$$

$\therefore$ NO STIFFENERS ARE REQUIRED FOR WEB CRIPPLING OF WALE

<u>CHECK WEB CRIPPLING OF SOLDIER BEAM</u>

N = 2(1+.67) = 3.34"

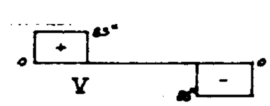
$$\frac{R}{t(N + 2_K)} = \frac{83}{.390(3.34 + 2(1.3125))} = 35.7\text{KSI} > 32.4\text{KSI N.G.}$$

$\therefore$ Stiffeners are required for web crippling of soldier beam, locate stiffeners on wale for ease of fabrication.

<u>CHECK WEB SHEAR IN WALE</u>

ALLOWABLE SHEAR STRESS $= 1.2 * (14.3KSI) = 17.4KSI$

$VMAX = 83*$



$$f_v = \frac{V}{dt} = \frac{83}{26.91(.450)} = 6.29KSI \angle 32.4KSI \ \underline{OK}$$

*INCREASE IN ALLOWABLE UNIT STRESS FOR TEMPORARY STRUCTURES

DESIGN EXAMPLE I (CON'T)

VI. <u>DESIGN OF STRUT</u>

$P = 2(R_L) = 2(83) = 166^K$

ASSUME FOR DESIGN EXAMPLE:

WIDTH OF CUT = 40 '

<u>NOTE:</u> PROVIDE LATERAL BRACING OF COMPRESSION FLANGE AT INTERVALS SUCH THAT A REDUCTION IN F_b IS NOT REQUIRED.

<u>TRY W10x54</u>

$L_u = 28.4 \quad \therefore$ PROVIDE LATERAL BRACING AT MID - POINT

$$f_a = \frac{P}{A} = \frac{166}{15.9} = 10.44_{KSI}$$

$$\frac{KL}{r_y} = \frac{20(12)}{2.56} = 93.8 \qquad \frac{KL}{r_x} = \frac{40(12)}{4.39} = 109.3$$

$\therefore F_a = 11.77_{KSI}$ (TABLE 1 - 36, pg. 5 - 84)

$$\frac{f_a}{F_a} = \frac{10.44}{11.77} = .887 > .15 \therefore \text{ USE AISC FORMULA (1.6 - 1a)}$$

CONSIDER D.L. MOMENT

$$M = \frac{w1^2}{8} = \frac{.054(40)^2}{8} = 10.8_{K-FT.} = 129.6_{K-in}$$

$$\Delta = \frac{5w1^4}{384EI} = \frac{5(.054)(40)^4(1728)}{384(29{,}000)(306)} = 0.35''$$

M DUE TO $P(\Delta) = 166(.35) = 58.1_{K-in} \qquad M_{TOTAL} = 187.7_{K-in}$

$$f_b = \frac{M}{S_x} = \frac{187.7}{60.4} = 3.11_{KSI}$$

$L_u > L_b = 20 \qquad \therefore F_b = .6F_y(1.2)^* = 25.9_{KSI}$

$$\frac{f_a}{F_a} + \frac{C_m f_b}{(1 - f_a / F'_e)F_b} \le 1 \qquad (1.6 - 1a)$$

$C_m = 1.0 \qquad F'_e = 12.50 \qquad$ (TABLE 2, pg. 5 - 94)

$$.887 + \frac{1.0(3.11)}{(1 - 10.44/12.50)25.9} = .887 + .729 = 1.616 \ \underline{\underline{\text{N.G.}}}$$

*INCREASE IN ALLOWABLE UNIT STRESS FOR TEMPORARY STRUCTURES

DESIGN EXAMPLE I (CON'T)

VI. DESIGN OF STRUT (Con't)

$\therefore$ USE SAG BARS LOCATED AT CENTER OF EACH STRUT AND SUPPORTED FROM DECKING.

THE SAG BAR REDUCES THE SIMPLE SPAN DEAD LOAD DEFLECTION BY 78.7% OR

$$\Delta = .213\ \Delta_{SIMPLE}$$

$$\therefore\ \Delta = .213\,(.35) = .075''$$

MAXIMUM DEAD LOAD MOMENT IN STRUT $= \dfrac{16\ wL^2}{512}$

$$\therefore M = \frac{16\,(.054)(40)^2}{512} = 2.7\ \text{K-Ft.} = 32.4\ \text{K-in}$$

L = LENGTH OF STRUT

M DUE TO P (Δ) = 166 (.075) = 12.45 K-in $\quad M_{TOTAL}$ = 44.85 K-in

$$f_b = \frac{M}{S_x} = \frac{44.85}{60.4} = 0.743\ \text{ksi.}$$

$$\frac{KL}{r_y} = 93.8 \qquad \frac{KL}{r_x} = \frac{20(12)}{4.39} = 54.7$$

$\therefore$ F_a = 13.74 ksi (TABLE 1-36, PAGE 5-84)

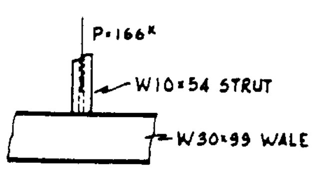
$$\frac{f_a}{F_a} + \frac{C_m\ f_b}{(1 - f_a / F'_e)\ F_b} \leq (1.6 - 1a)$$

C_m = 1.0 $\qquad F'_e$ = 49.92 (TABLE 2, PAGE 5-94)

$$\frac{10.44}{13.74} + \frac{1.0(.743)}{(1 - 10.44 / 49.92)25.9} = 0.760 + .036 = .796 < 1 \quad \text{o.k.}$$

$\therefore$ USE W 10 x 54 OR EQUIVALENT FOR STRUT

CHECK WEB CRIPPLING OF WALE



$$\frac{R}{t\,(N + 2k)} \leq .75\ F_y\,(1.2)\ *$$

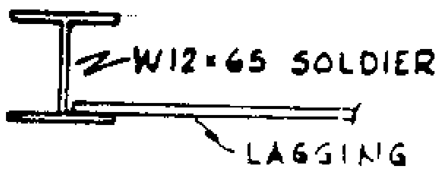
WHEN WEBS OF INTERSECTING MEMBERS ARE PERPENDICULAR TO EACH OTHER, STIFFNERS ARE REQUIRED TO TRANSFER STRESS.

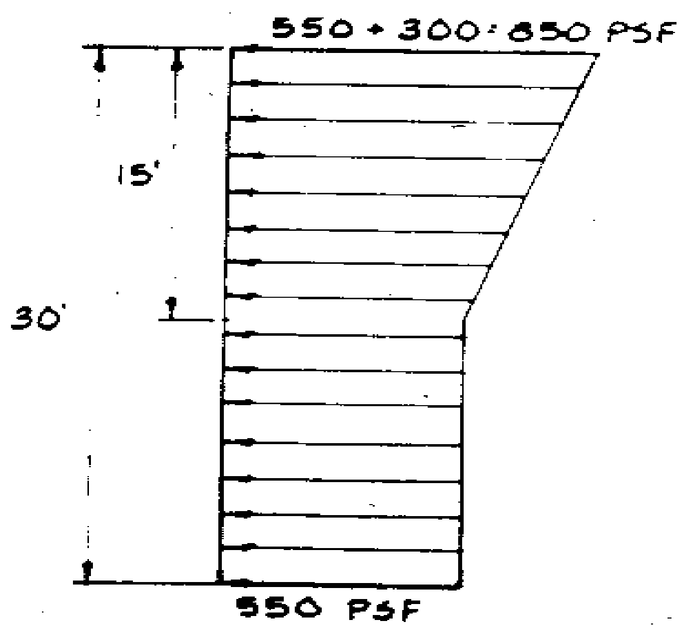
DESIGN EXAMPLE I (CON’T)

VII <u>DESIGN OF TIMBER SHEETING</u>

FOR DESIGN OF LAGGING ASSUME POINT T OF SUPPORT MIDWAY BETWEEN FLANGE (SEE NOTE 4 OF GENERAL PROVISIONS)

CONSTRUCTION EQUIPMENT ASSUMED TO BE ADJACENT TO CUT, THEREFORE DESIGN LAGGING FOR FULL LATERAL EARTH PRESSURE PLUS SURCHARGE. (SEE NOTE ON LP-18)





LATERAL EARTH PRESSURE & SURCHARGE
(SEE SHEET LP-27)

<u>ALLOWABLE STRESSES</u>
NATIONAL DESIGN SPECIFICATION FOR STRESS-GRADE LUMBER AND ITS FASTENINGS, NATIONAL FOREST PRODUCTS ASSOCIATION, 1968

FROM SUPPLEMENT TO 1968 EDITION TABLE 1 DENSE STRUCTURAL GRADE (DOUGLAS FIR)

3” ¢ 4” F_b = 1980 PSI

5” F_b = 1900 PSI

AN INCREASE OF 50% IN THE ALLOWABLE BENDING STRESSES IS ALLOWED FOR <u>NEW</u> LUMBER OR OLD LUMBER INSPECTED AND APPROVED BY THE ENGINEER.

<u>DESIGN LAGGING FOR NEW LUMBER</u>

3” ¢ 4” F_b = 1.5 (1930) = 2925PSI

5” F_b = 1.5 (1900) = 2850PSI

DESIGN EXAMPLE I (CON'T)

$$M = \frac{w\ell^2}{8} x \frac{w\,(1)\,(9.48)^2}{8} = 11.23w \text{ Ft - lb}$$

$$S = \frac{bd^3}{12} x \frac{2}{d} = \frac{12d^3(2)}{12d} = 2d^2 \,(in^2)$$

$$Fb = \frac{M}{S}$$

FOR 3" & 4" SHEETING: $2925 = 67.38 \frac{w}{d^2}$

$w = 43.41\,(d^2)$ PSF

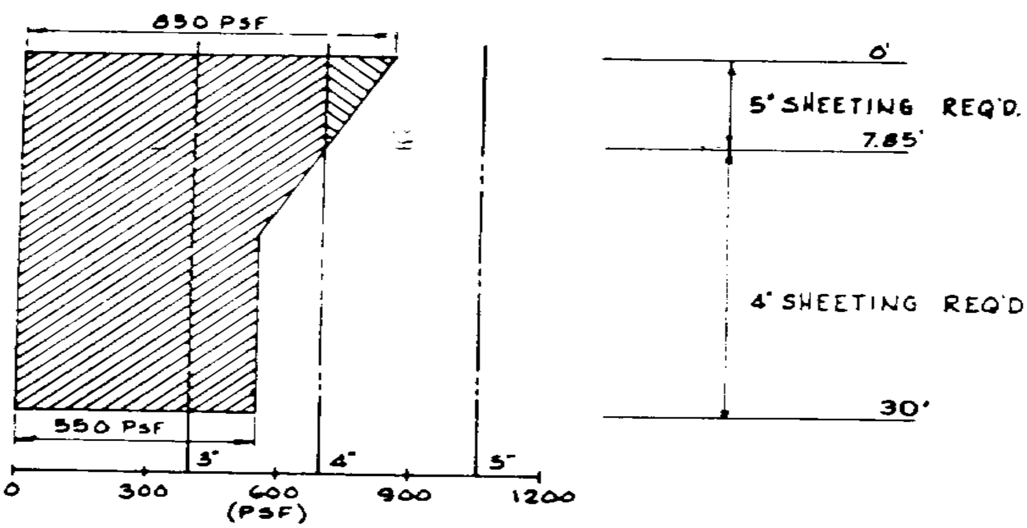
$$= \frac{11.23\,w\,(12)}{2d^2}$$

FOR 5" SHEETING: $2850 = 67.38 \frac{w}{d^2}$

$w = 42.30\,(d^2)$ PSF

$$Fb = \frac{67.38w}{d^2}$$

MAX. w FOR 5" x 12" = 1057 PSF
MAX. w FOR 4" x 12" = 694 PSF
MAX. w FOR 3" x 12" = 390 PSF



SHEETING SCHEDULE

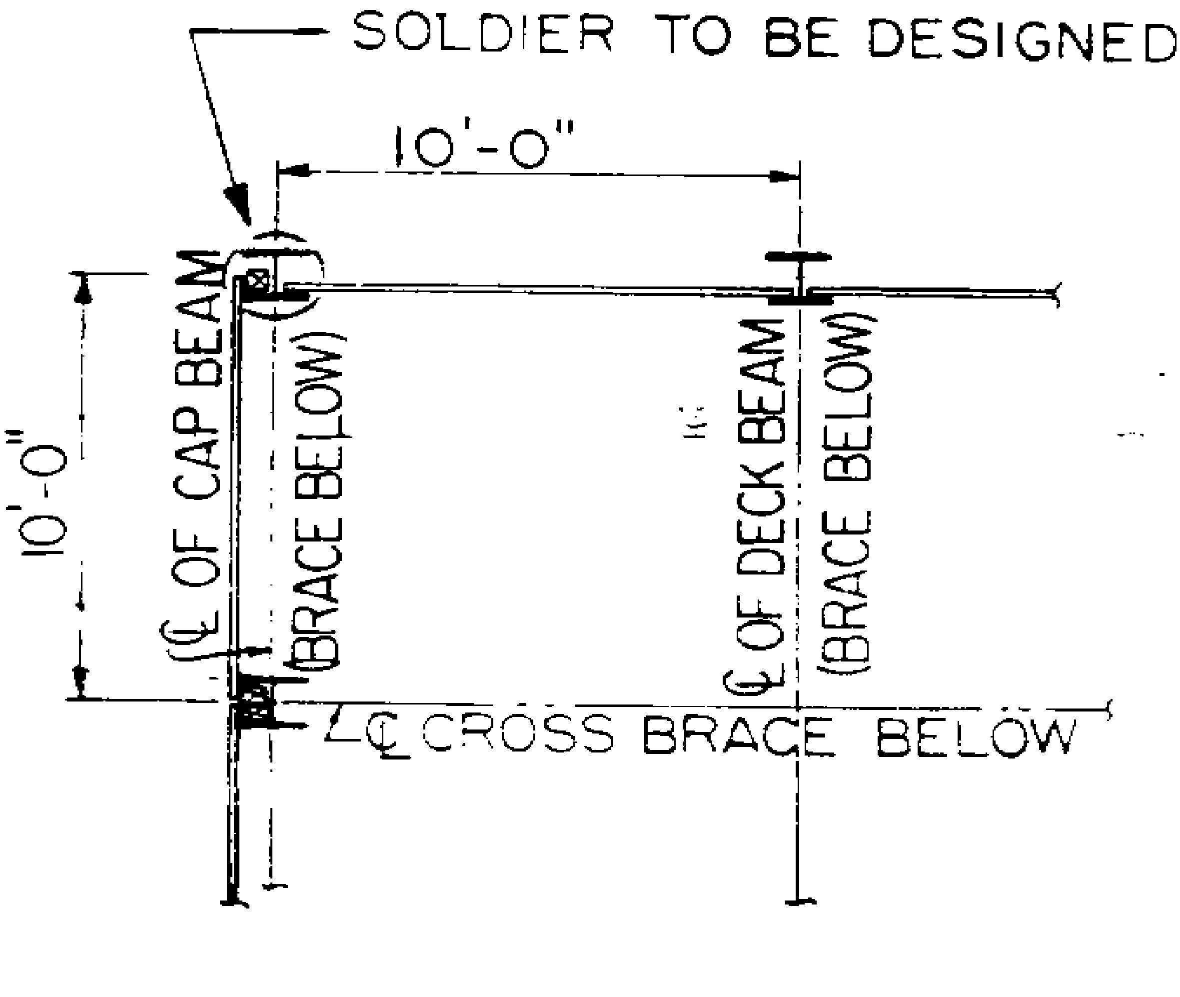
FOR NEW DENSE STRUCTURAL GRADE DOUGLAS FIR:

USE 5" x 12" 0' TO 8'
USE 4" x 12" 8' TO 30'

DESIGN EXAMPLE Ia

SOLDIER BEAM FOR BIAXIAL BENDING
AND AXIAL COMPRESSION

SIMILAR TO DESIGN EXAMPLE I EXCEPT THE SOLDIER BEAM IS SUBJECTED TO BIAXIAL BENDING AND AXIAL COMPRESSION



DESIGN EXAMPLE Ia

DESIGN OF SOLDIER BEAM

$P = 50^k$

M_{XX} = 16.875 (5) = 84.35^{k-1}
M_{YY} = 16.875 (5) = 84.35^{k-1}
Lx = 14 FT.
Ly = FULLY BRACED (SOLDIER IS DRIVEN INTO SOIL, NOT PITTED)

TRY W14x90 (COMPACT SECTION IN A-36)
A = 26.5 in^2
Sx = 143 in^3
Sy = 49.9 in^3 pg.1-22, AISC 8th EDITION
r_x = 6.14 in
r_y = 3.70 in

f_a = P/A = 50/26.5 = 1.89 ksi
$f_{bx} = (M/S)_x$ = 84.35 (12)/143 = 7.08 ksi
$f_{by} = (M/S)_y$ = 84.35 (12)/49.9 = 20.28 ksi

FOR TEMPORARY STRUCTURES Fb = 1.2 Fb (AISC)
Fbx = 1.2 (24) = 28.8 ksi pg. 5-20
Fby = 1.2 (27) = 32.4 ksi pg. 5-21

Kl/r_x = (14)(12)/6.14 = 2.74
$\therefore$ Fa = 20.12 ksi (TABLE 3-36, pg. 5-74)
fa/Fa = 1.89/20.12 = 0.09<0.15
$\therefore$ USE AISC FORMULA (1.6-2)

DESIGN EXAMPLE Ia (CONT'D)

DESIGN OF SOLDIER BEAM

$fa/Fa + fbx/Fbx + fby/Fby \leq 1$

$= 0.09 + 7.08/28.8 + 20.28/32.4$

$= 0.09 + 0.246 + 0.626 = 0.96 < 1$ OK

$\therefore$ USE W14x90 OR EQUIVALENT FOR SOLDIER BEAM

BALANCE OF DESIGN SIMILAR TO DESIGN EX. I

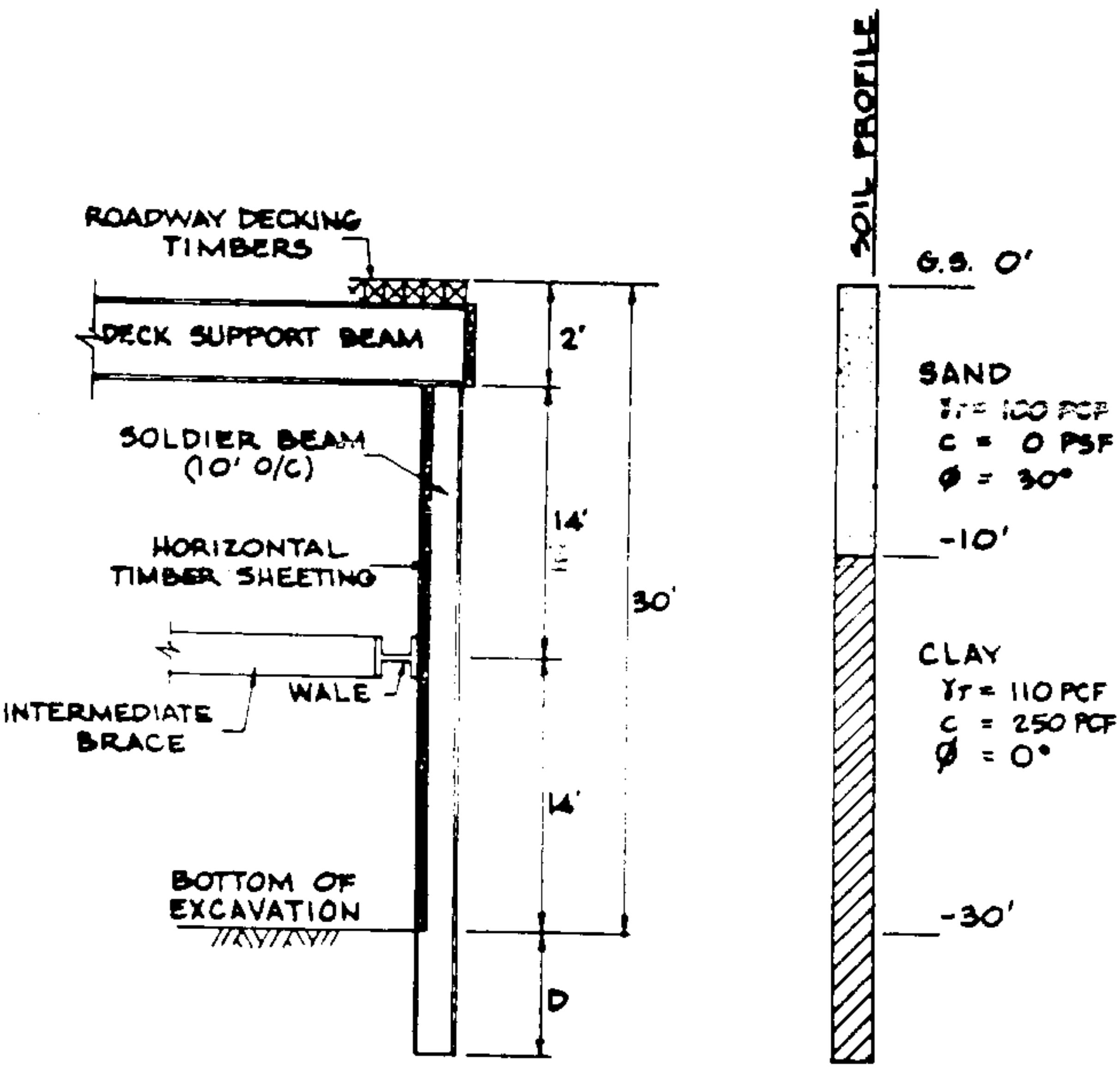
LATERAL EARTH PRESSURE DISTRIBUTION FOR

FOR

TEMPORARY EARTH RETAINING STRUCTURES

DESIGN EXAMPLE II - STRATIFIED SOIL

DESIGN EXAMPLE II



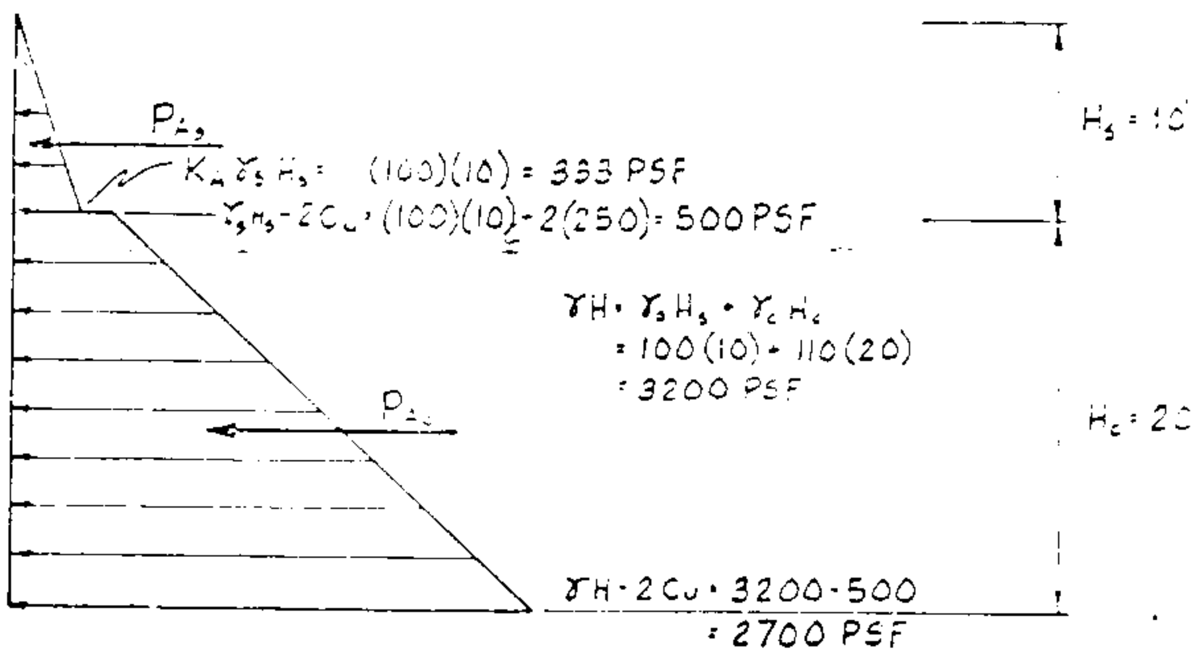
DESIGN EXAMPLE II (CON'T)

GIVEN:
- The soldier beam and horizontal timber sheeting temporary earth retaining structure shown on page LP-40.
- The given soils profile and design parameters as interpreted from boring information.

DETERMINE: LATERAL PRESSURES

Note: Determination of design loads and procedures for design of soldier beam, wale, intermediate brace and horizontal timber sheeting similar to Design Example I.

ORIGINAL PRESSURE DISTRIBUTION

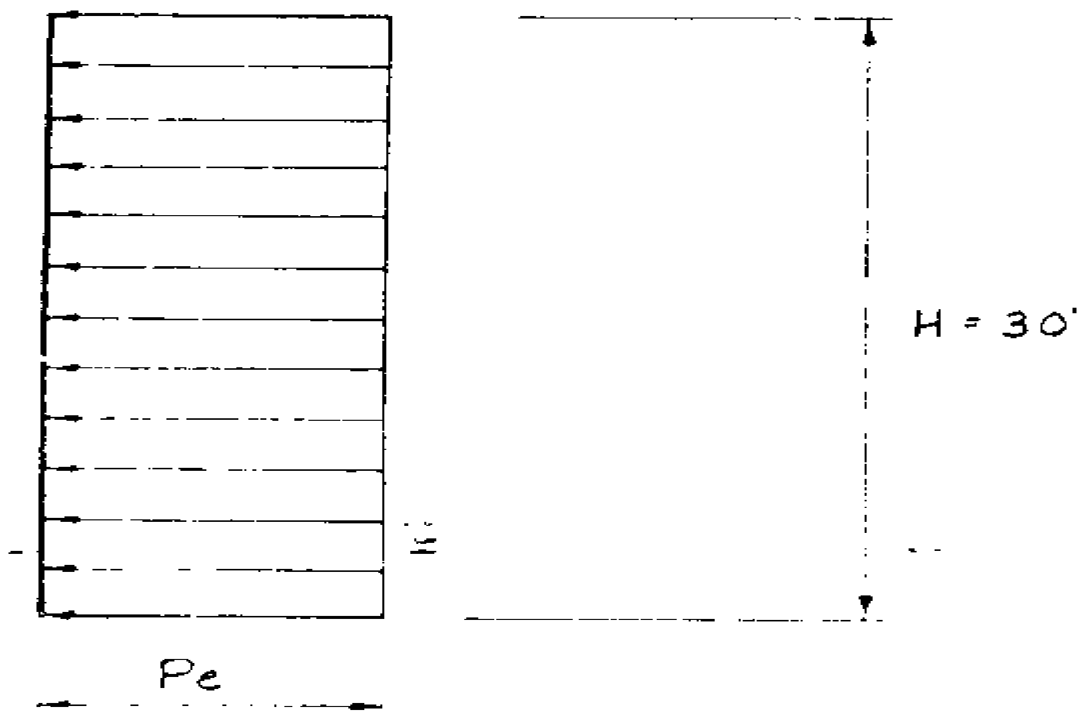


$$P_{A_s} = \frac{1}{2}(333)(10) = 1665 \ \text{lb/Ft.}$$

$$P_{A_c} = \frac{20}{2}(500 + 2700) = 32{,}000 \ \text{lb/Ft.}$$

DESIGN EXAMPLE II (CON'T)

REDISTRIBUTION



$$Pe = \frac{(\sum PA_S + \sum PA_C}{H}$$

$$= \frac{1665 + 32000}{30}$$

$$Pe = 1122 \text{ PSF}$$

FOR 10' SPACING OF SOLDIER BEAMS

$$Pe = \underline{\underline{11.2 \text{ Kips / Ft.}}}$$

BALANCE OF DESIGN SIMILAR TO DESIGN EXAMPLE I

SECTION DS

ROADWAY & SIDEWALK DECKING

SYSTEMS

LOADS AND CRITERIA FOR DESIGN OF STEEL DECKING SUPPORT BEAMS

1. Allowable unit stresses for decking support beams shall be as specified in Section AS, Allowable Unit Stresses for Temporary Structures & Underpinning.

2. The dead load shall include the weight of pipes and other subsurface structures carried by the decking in addition to the dead load of the decking system itself.

3. The live load shall be computed in either of the following ways:

   a) 250 pounds per square foot on any two adjacent 10 foot roadway lanes and 200 pounds per square foot on the remaining area of roadway and sidewalk.

   b) On any two adjacent 10 foot roadway lanes, a single group of four resultant wheel loads of 31,000 pounds each with a consecutive spacing of 6, 4, 6, feet placed on line directly over the decking support beams and at right angles to the direction of traffic, and at the center of each 10 foot by 10 foot area of roadway and sidewalk outside of these lanes, two resultant wheel loads of 10,000 lbs. each placed 6 feet apart.

   If less than two 10 foot traffic lanes are available, place a single group of two wheel loads of 31,000 pounds with a spacing of 6 feet in the available traffic lane.

   The 31,000 pound load is the resultant of the two rear wheel groups of a 15 yd. Concrete Truck, having a spacing of 4.5 feet between their rear axles and is based on a decking support beam spacing of 10 feet center to center. Using the 15 yd. Concrete Truck wheel loading shown in Figure #1, the 31,000 pound wheel resultant can be modified for a closer decking support beam spacing.

4. The previous loads, both uniform and concentrated, shall be placed so as to produce the maximum bending moment for each case. The design of the decking support beam is to be based on the more critical moment.

5. The decking support beam must be checked for web shear capacity using the maximum reaction on the decking support beam.

   To determine the axial components in the design of the soldier beam and bending moment in the strong axis of the cap beam, the loads should be positioned to produce the maximum reaction on the decking beam support system.

6. The allowable maximum total load deflection of a decking support beam shall be 1/240 of its clear span; however, it is subject to review when the working drawings are submitted by the contractor and will be considered with reference to utilities supported via decking and other factors that impact deflection. The maximum total load deflection shall be calculated using the reduced moments and shears as indicated in paragraph 9, page DS-3.

7. Decking Support Beams shall have a maximum spacing of ten (10) feet center to center. The use of a greater spacing will be permitted only with the approval of the Engineer.

8. Where the loading due to the Contractor's machinery or equipment is in excess of paragraph 3, page DS-2, the street and sidewalk supporting system shall be of sufficient strength to safely support such loads.

9. Reduction of Load Intensity in Design of Decking Support Beams.

   Where maximum moments and shears are produced in any member by loading any number of traffic lanes simultaneously, the following percentages of the live load moments and shears shall be used in view of the improbable coincidence of loading =

One Lane	100%
Two or Three Lanes	90%
Four or More Lanes	85%

TIMBER DECKING REQUIREMENTS

1. Allowable unit stresses for timber decking shall be as specified in Section AS, "Allowable Unit Stresses for Temporary Structures and Underpinning".

2. Wherever possible, roadway decking timbers should span between at least three decking support beams.

   Roadway decking timbers shall be structural grade, with a minimum of 12" x 12" (full size) for a ten (10) ft. span. Used timber may be allowed with approval of Engineer.

3. Roadway decking timbers spanning between two decking support beams will be permitted on a limited basis to provide access hatches for excavation equipment or as otherwise required for the contractor's operations.

4. Wherever possible, roadway decking timbers shall be placed with the twenty (20) foot dimension parallel to the direction of the traffic.

5. Roadway and sidewalk decking shall conform to the general elevation of the permanent roadway and sidewalk they are temporarily replacing.

6. Decking timbers shall be designed for maximum moment and shear resulting from two 10,000 pound wheel loads spaced 4' - 6" apart. This loading is based on decking timbers running parallel to the direction of traffic. At intersections and other special locations, the decking timber design load should be evaluated and based on the wheel loading of a 15cy. Concrete truck as given on page DS-7. Also see paragraph 8, page DS-3, for heavy equipment.

CONCRETE DECKING PANELS

1. Precast reinforced concrete panels may be substituted for timber decking for portions of the work with the approval of Engineer.

2. Allowable unit stresses for concrete decking panel shall be as specified in Reference Standard RS-3.

3. Concrete panels shall be designed for maximum moment and shear resulting from two 20,000 pound loads spaced 4'-6" apart, positioned for maximum moment and maximum shear.

4. Concrete panels to be properly anchored to decking support beams.

5. Minimum thickness of concrete panels shall be 9". The preferred thickness is 12".

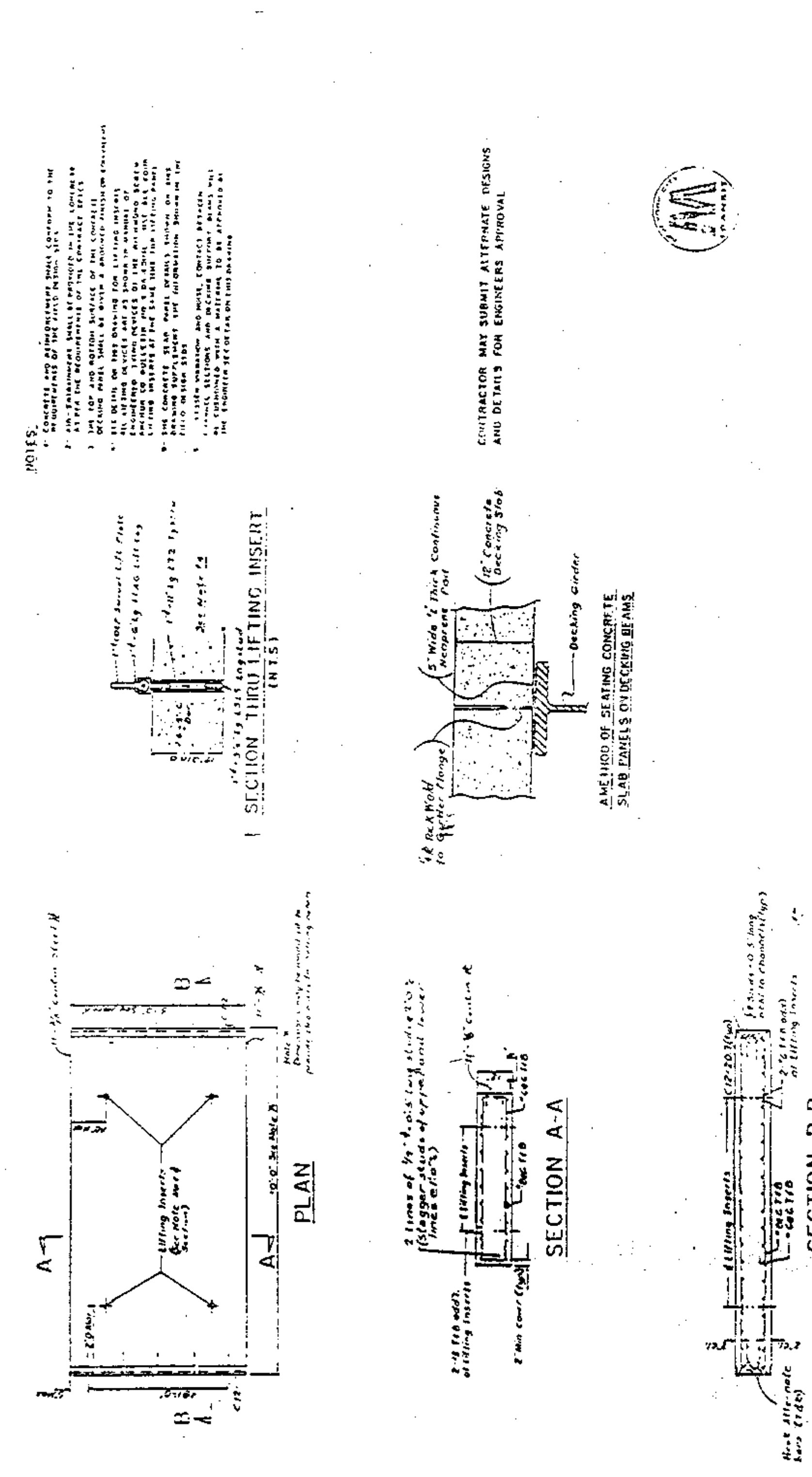
6. Panels designed of prestress concrete may be submitted to the Engineer for approval.

7. The concrete panel design loading and minimum thickness are based on the assumption of a 5' x 10' concrete panel used in conjunction with a 10' spacing of decking support beams. The 10' length of the concrete panel is assumed to be parallel to the direction of traffic. Concrete panels used at intersection and other special locations should be designed based on their location and the wheel loading of a 15cy. Concrete truck as given on page DS-8.

8. The wearing surface of each panel to be broom finished.

9. Reuse of panels to be approved by Engineer due to potential for wear and weathering.

   A Typical Concrete Decking Panel Drawing is shown on Page DS-6A.



TYPICAL CONCRETE DECK SLAB PANEL

RAMPS

Where existing street surface elevation must be raised to clear utilities, ramps subject to approval of the Engineer may be used. The live load for the decking support beams under the ramp itself and the first twenty (20) feet of horizontal decking following the ramp shall be increased by 30% to account for impact.

SLOPES

When designing decking support beams, where the existing street surface is on a slope, the lateral component of the decking must be provided for. The design shall use adequate bracing to stabilize the support system.

STEEL PLATES OVER NARROW EXCAVATIONS

Steel plates are generally a standard size of 5' x 10'. The plate thickness and corresponding maximum clear spans are given below as a guide. The contractor may submit other plate thicknesses and/or clear spans for approval by the Engineer. The plates are assumed to be placed with the 10' dimension perpendicular to the trench excavation. The plates should be spiked and ramped with asphalt concrete or otherwise secured to prevent movement due to the vibrations caused in the plate from the flow of traffic. Steel plates used for pedestrian traffic shall be provided with a non-slip surface.

	PLATE SIZE	MAXIMUM CLEAR SPAN (Ls)
ONE WAY SUPPORTED		
	1"	3' - 0"
	1¼"	4' - 0"
	1½"	5' - 0"
	1¾" OR TWO 2¼"	6' - 0"
	2" OR ONE 1-½" PLUS 1¼"	8' - 0"
TWO WAY SUPPORTED	¾"	4' - 0" x 3' - 0"
	1"	6' - 0" x 3' - 0"
	1"	8' - 0" x 3' - 0"

WHEEL LOADING 15cy CONCRETE TRUCK GROSS WT. 50$^{\underline{TON}}$

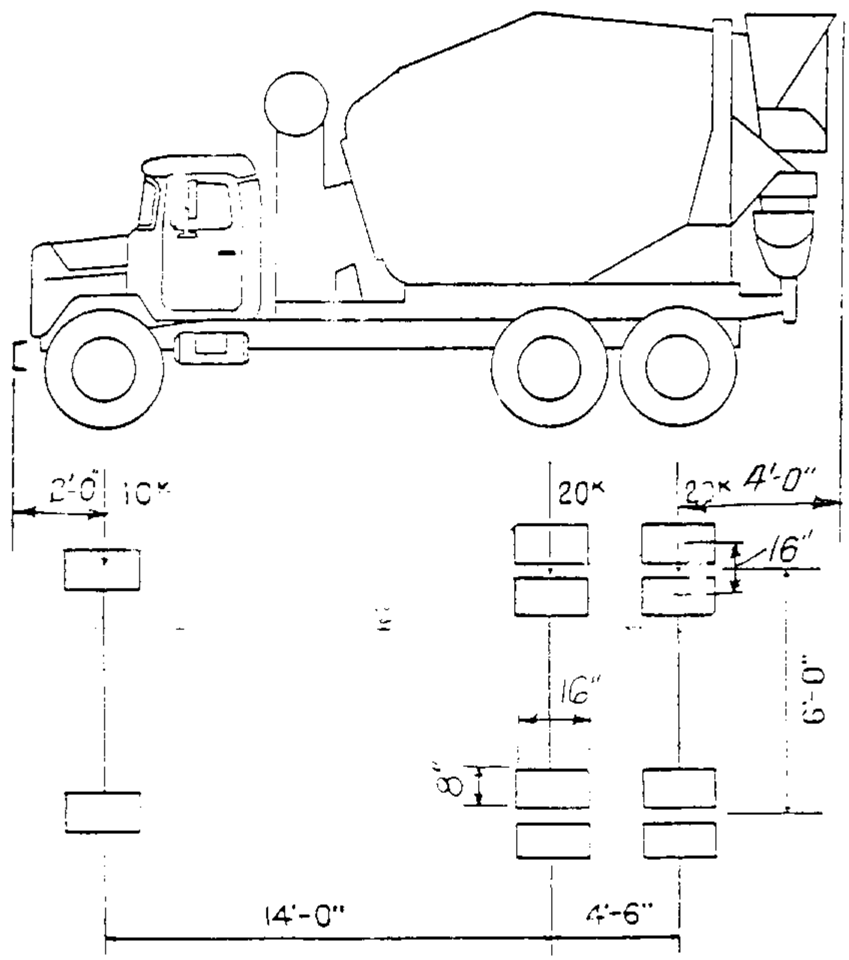


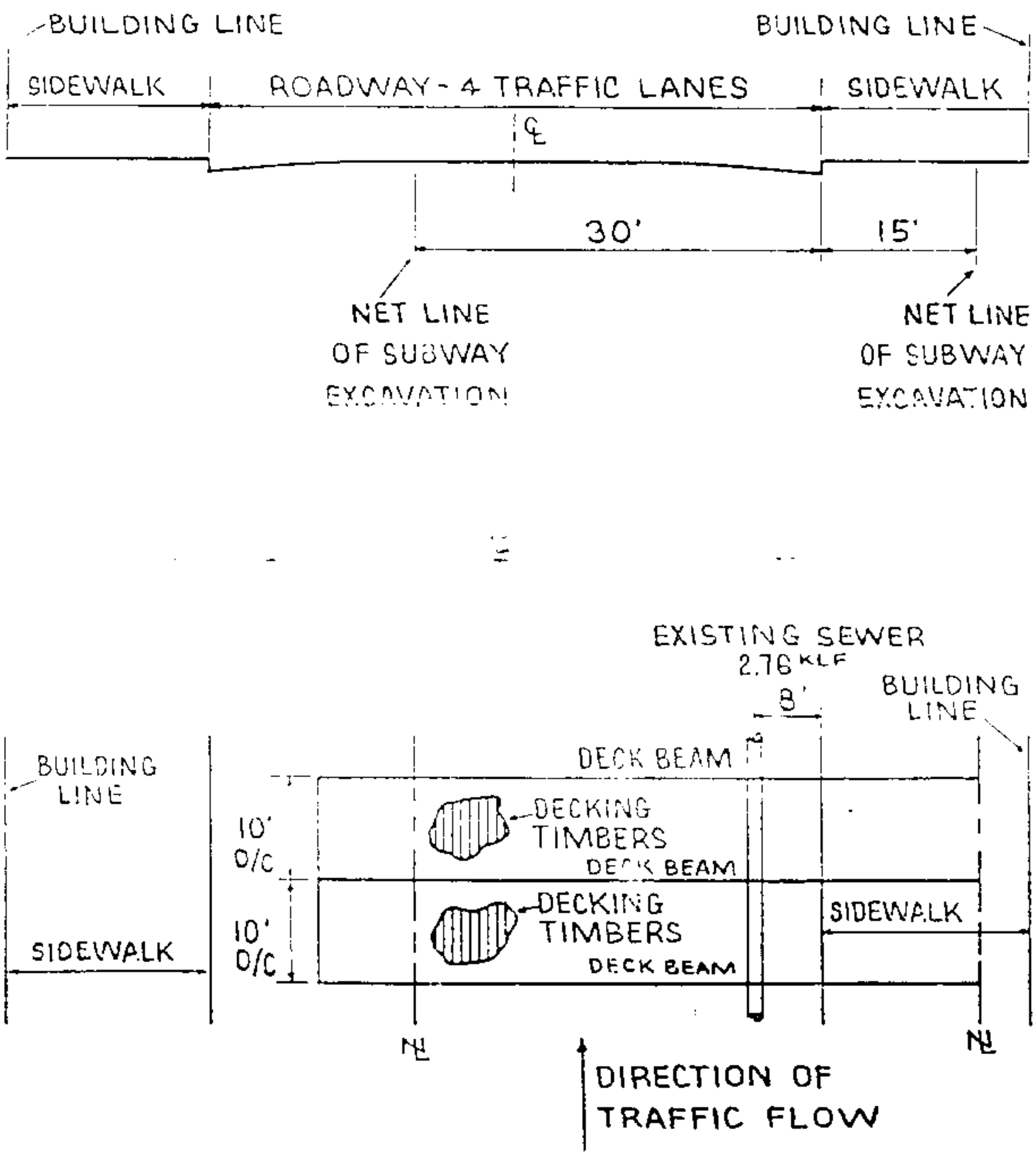
FIGURE 1

WHEEL LOAD = 10ᴷ/WHEEL
RESULTANT OF TWO REAR WHEELS = 20ᴷ
FRONT AXLE LOAD = 20ᴷ
REAR AXLE LOAD = 40ᴷ EACH

ROADWAY & SIDEWALK
DECKING SYSTEMS

DESIGN EXAMPLE III
DECKING SUPPORT BEAM

DESIGN EXAMPLE III



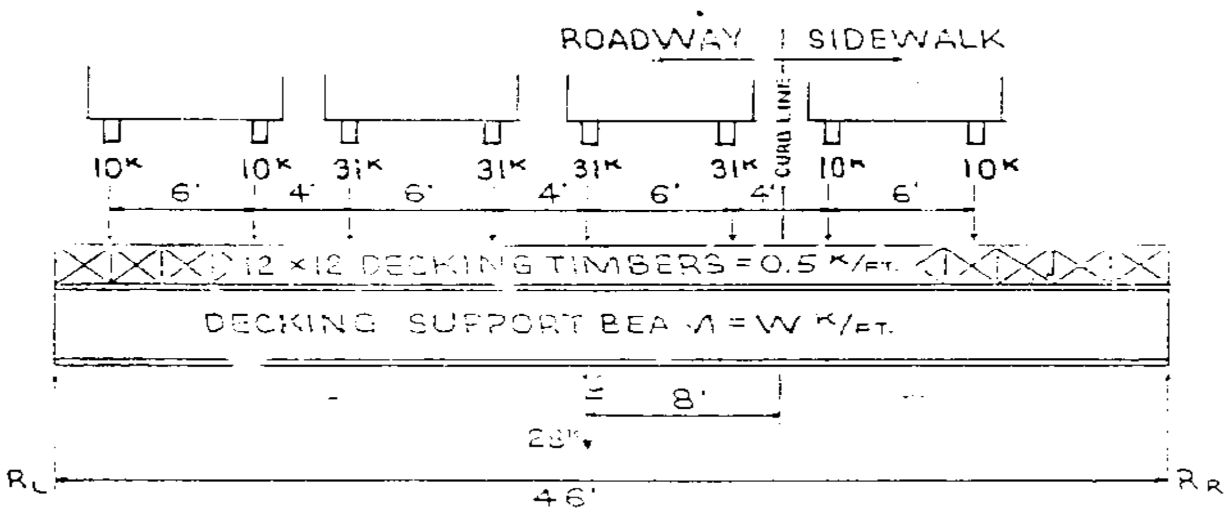
DESIGN OF DECKING SUPPORT BEAM

SPACING OF DECKING SUPPORT BEAM = 10' O/C
DESIGN SPAN = 45' +d*

* d = depth of soldier beam

ASSUME W14 x 84 SOLDIER BEAM FOR THIS DESIGN
EXAMPLE d = 14.18" (AISC 7TH Ed. p. 1-36)

$$\text{DESIGN SPAN} = 45' + \frac{14.18}{12} = 46.18' \quad \text{SAY } 46'$$



<u>DETERMINE CENTROID OF WHEEL LOADING SYSTEM</u>

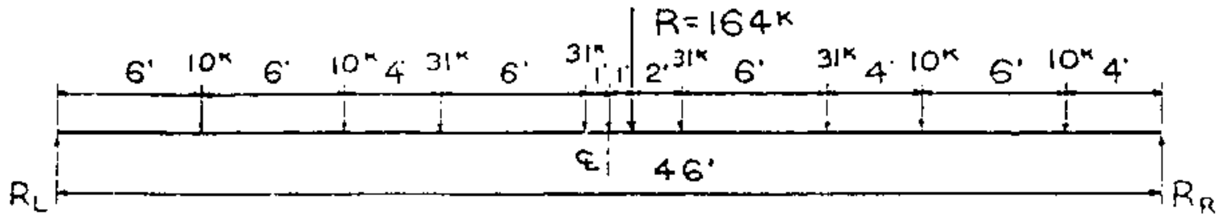
10 x 0 = 0
10 x 6 = 60
31 x 10 = 310

$$\bar{x} = \frac{2952}{164} = 18' \text{ FROM LEFT } 10^K \text{ WHEEL}$$

31 x 16 = 496
31 x 20 = 620
31 x 26 = 806
10 x 30 = 300
10 x 36 = 360
164 2952

DESIGN EXAMPLE III (CON'T)

POSITION WHEEL LOADS FOR MAXIMUM MOMENT

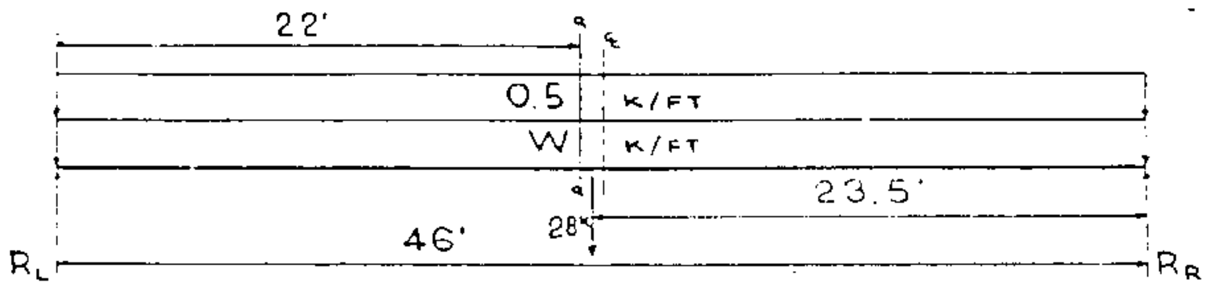


$R_L = 22/46\ (164) = 78.4^K \uparrow$
$R_R = 24/46\ (164) = 85.6^K \uparrow$

MAXIMUM LIVE LOAD MOMENT

$M_{LL} = 78.4(22) - 31(6) - 10(10) - 10(16) = 1724.8 - 186 - 100 - 160$
$M_{LL} = 1278.8\text{K - FT}$
APPLY MOMENT REDUCTION AS PER ITEM 9, pg. DS-3
$\therefore$ REDUCTION FACTOR = 90%
$M_{LL} = .9(1278.8) = 1150.9\text{K - FT}$

NOTE: FOR EASE OF COMPUTATION, DEAD LOAD COMPONENT OF DESIGN MOMENT SHOULD BE COMPUTED AT SAME POSITION AS MAX. LIVE LOAD MOMENT



$$R_L = \frac{23}{46}(0.5)(46) + \frac{23.5}{46}(28) + \frac{23}{46}(w)\ (46) = 11.5 + 14.3 + 23w$$
$R_L = 25.8 + 23w \uparrow$
$$R_R = \frac{23}{46}(0.5)(46) + \frac{22.5}{46}(28) + \frac{23}{46}(w)(46) = 11.5 + 13.7 + 23w$$
$R_R = 25.2 + 23w \uparrow$
$Ma\text{-}a = (25.8 + 23w)\ (22) - 0.5(22)\ (11) - w(22)\ (11) =$
$567.6 + 506w - 121 - 242 = 446.6 + 264w\ \text{K - FT}$

DESIGN EXAMPLE III (Con't)

DESIGN MOMENT

$M = 1150.9 + (446.6 + 264W) = 1597.5 + 264W$ K - FT

PROVIDE BLOCKING AND TIE RODS AT INTERVALS SUCH THAT THE COMPRESSION FLANGE CAN BE CONSIDERED LATERALLY SUPPORTED FOR $F_b = .66F_y = 24$KSI

$\therefore$ ALLOWABLE BENDING STRESS AS PER SPECIAL PROVISION 2, pg. AS - 2

$= 1.2(24) = 28.8$KSI

FROM SOLDIER BEAM CALCULATIONS, AXIAL LOAD ON DECK BEAM,

$P_{DB} = 70^K$ (Assumed for this design example)

TRY W33 x 220 $\left\{\begin{array}{ll} A = 64.8 \text{ in}^2 & r_x = 13.8 \text{ in} \\ S_x = 742 \text{ in}^3 & r_y = 3.6 \text{ in} \end{array}\right\}$ AISC 7TH Ed. pg. 1 - 28

$\ell c = 16.7$ft pg. 2 - 30

$f_a = P/A = 70/64.8 = 1.08$KSI

PROVIDE BLOCKING AND TIE RODS AT THIRD POINTS

$\therefore$ ACTUAL UNBRACED LENGTH

$= 46/3 = 15.33$ ft < 16.7

$K\ell/r_x = 46(12)/13.8 = 40$

$\left\{\begin{array}{lll} 52 & 18.17 & \\ 51.1 & \longrightarrow & F_a = 18.25\text{KSI} \\ 51 & 18.26 & \text{Table1 - 36 pg. 5 - 84} \end{array}\right.$

$K\ell/r_y = 15.33(12)/3.6 = 51.1$

$f_a/F_a = 1.08/18.25 = 0.059 < .15$ $\therefore$ USE AISC FORMULA 1.6 - 2 (pg. 5 - 22)

$f_a/F_a + f_b/F_b \leq 1$

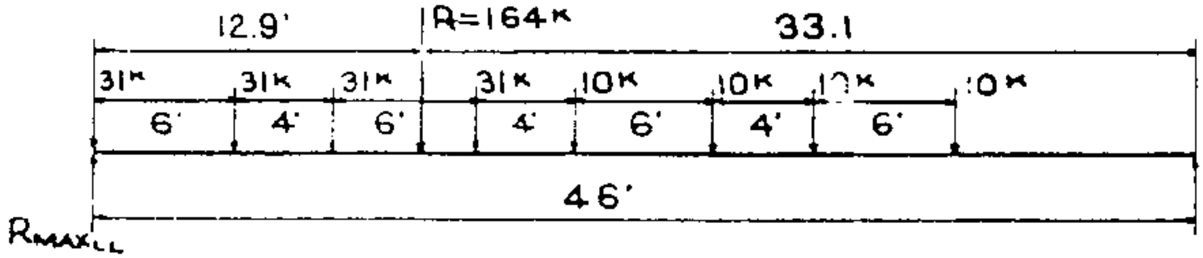
$f_b = M/S_x = (1597.5 + 264(.22))(12)/742 = 26.77$KSI

$0.059 + 26.77/28.8 = .059 + .930 = .989 < 1$ O.K.

DESIGN EXAMPLE III (CON'T)

DETERMINE MAXIMUM REACTION

POSITION WHEEL LOADS FOR MAXIMUM REACTION



DETERMINE CENTROID OF WHEEL LOADING SYSTEM

31 x 0 = 0
31 x 6 = 186
31 x 10 = 310
31 x 16 = 496
10 x 20 = 200
10 x 26 = 260
10 x 30 = 300
10 x 36 = 360
164 2112

$$\bar{x} = \frac{2112}{164} = 12.88' \text{ FROM LEFT } 31^K \text{ WHEEL}$$

$$R_{MAX_{LL}} = \frac{33.1}{4.6}(164) = 118^K$$

APPLY REDUCTION AS PER ITEM 9, pg. DS-3

$$\therefore R_{MAX_{LL}} = .9(118^K) = 106.2^K$$

REACTION COMPONENT DUE TO DEAD LOAD

$$R_{MAX_{DL}} = 25.8 + 23(.22) = 30.9^K$$
$$R_{MAX} = 106.2 + 30.9 = 137.1 \text{ KIPS}$$

NOTE: This value is to be used as the Axial Load on the Soldier Beam, P_{SB}

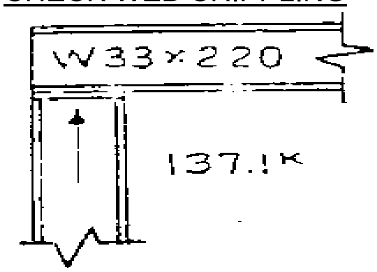
DESIGN EXAMPLE III (CON'T)

CHECK WEB SHEAR

$V_{MAX} = 137.1^K$

$f_v = V/dt = 137.1^K/(33.25)\,(.775) = 5.32_{KSI} < 1.2 * (14.5)$ O.K.

CHECK WEB CRIPPLING



$R/T\,(N + K) = .75\,F_y\,(1.2) *$

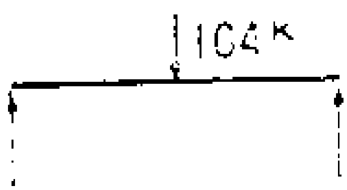
$137.1^K/.775\,(14.18 + 2.3125) = 10.73 < 32.44_{KSI}$ O.K.

CHECK DEFLECTION

ALLOWABLE MAXIMUM DEFLECTION = 46 X 12/2440 = 2.3"

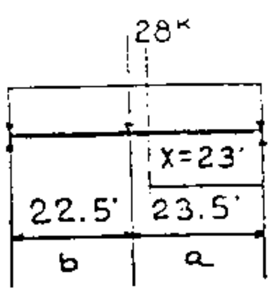
LIVE LOAD DEFLECTION

AS A FIRST TRY CONSIDER RESULTANT OF WHEEL LOADS AT MID-SPAN



$$\therefore\ \delta_{LL} = P\ell^3/48\,EI$$

$$\delta_{LL} = \frac{164^K\ (46)^3\,\cancel{ft}^3\ x\ 1728\,\dfrac{in^3}{\cancel{ft}^3}}{48\left(29000\ \dfrac{\cancel{k}}{\cancel{in}^2}\right)(12300in^4)} = 1.61\ in$$



$$\delta_u = \frac{5W\ell^4}{384EI} = \frac{5\left(.72\ \dfrac{\cancel{k}}{\cancel{ft}}\right)\left(\dfrac{1\,\cancel{ft}}{12\,\cancel{in}}\right)(46)^4\left(\cancel{ft}^4\right)\dfrac{20736\,in^{4}}{\cancel{1}\,\cancel{ft}^{2}}}{384\left(29000\ \dfrac{\cancel{k}}{\cancel{in}^2}\right)\left(12300\,in^{4}\right)}$$

$$\delta_u = .20\ in$$

$$\delta_p = \frac{Pbx}{6EI\ell}\left(\ell^2 - b^2 - x^2\right)$$

$$\delta_p = \frac{28^{\cancel{k}}(22.5)(23)12\,in/\cancel{ft}}{6\left(29000\ \dfrac{\cancel{k}}{\cancel{in}^2}\right)\left(12300\,in^{4}\right)(46)}\left(46^2 - 22.5^2 - 23^2\right)\frac{144\,in^2}{\cancel{ft}^2}$$

$$\delta_p = .27\ in$$

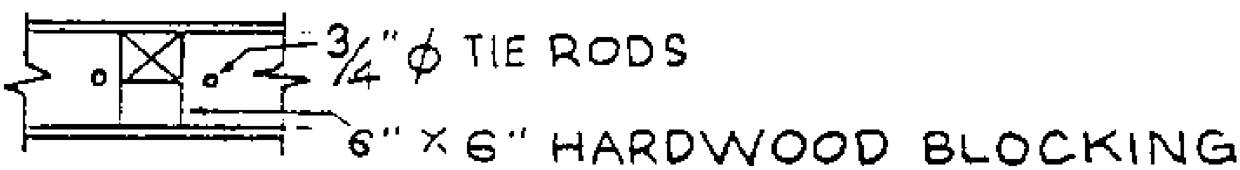
$$\delta_{CL} = 1.61 + .20 + .27 = 2.08" < 2.3" \qquad O.K.$$

*INCREASE IN ALLOWABLE UNIT STRESS FOR TEMPORARY STRUCTURES

DESIGN EXAMPLE III (CON'T)

NOTE: SHOULD THE DEFLECTION DETERMINED BY THIS APPROXIMATION EXCEED THE ALLOWABLE MAXIMUM DEFLECTION THEN A MORE EXACT METHOD, SUCH AS VIRTUAL WORK, CONJUGATE BEAM, ETC., SHOULD BE USED TO DETERMINE THE ACUTRAL MAXIMUM DEFLECTION OF THE DECKING SUPPORT BEAM.

CHECK SECONDARY LATERAL BRACING



DESIGN SECONDARY BRACING USING A MINIMUM AXIAL LOAD OF 2% OF THEJ LOAD IN THE PRIMARY BRACING MEMBER (SPECIAL PROVISION 6. pg. AS-3)

$$P = .02(70) = 1.4^{K}$$

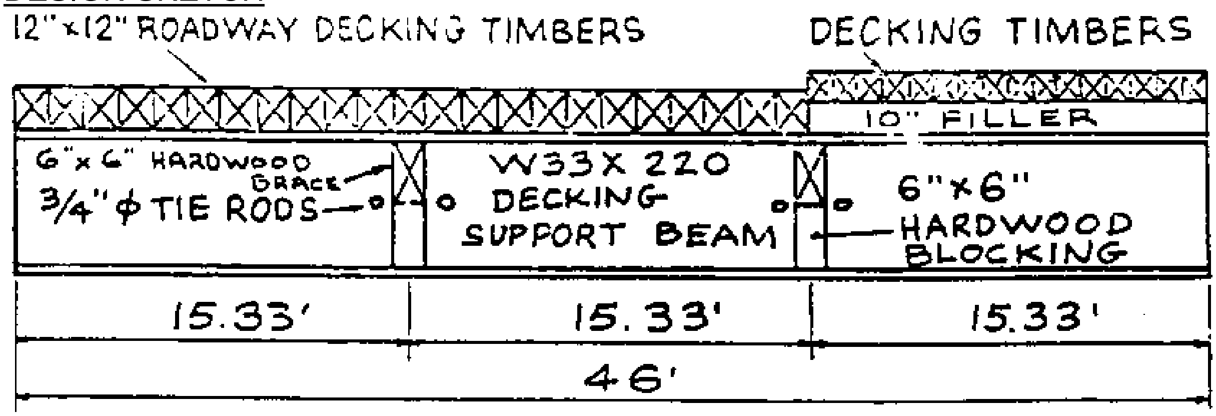
$$f_a = P/A = 1.4/36 = 38.9\text{psi}$$

$$F'c = 0.30\, E \Big/ (\ell/d)^2 \quad \text{RS - 4 SECT. 401 - E - 2 pg. IV - 5}$$

FOR DENSE STRUCTURAL GRADE DOUGLAS FIR E = 1,700,000psi
(SUPPLEMENT TO RS - 4, TABLE 1, pg. II - 5)

$$F'c = .30(1700000) \Big/ \left(\frac{10x12}{6}\right)^2 = 1275\text{psi} > f_a \qquad \text{O.K.}$$

DESIGN SKETCH



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SECTION UP

UNDERPINNING

UNDERPINNING

1. Loads and Stresses

For the determination of live load and dead load to be used in the design of underpinning, see Reference Standard RS-3, Chapter 3, and Section LS, pp. LS-1 through LS-12.

For the determination of allowable soil bearing pressures, see Reference Standard RS-9, Article 11, Section C-26-1103.4.

Allowable stresses to be used for steel, concrete, and timber are contained in Section AS.

2. DEFINITIONS

a) Underpinning

A method of construction that will permanently transmit the foundation loads of an adjacent structure to:

1. An appropriate lower soil level or stratum as is necessary to secure the structure from any settlements or lateral movements caused by construction operations.

2. An appropriate lower soil level or stratum capable of sustaining the soil pressure due to the original foundation loads and the manner in which these loads are transmitted by the underpinning.

3. An appropriate lower soil level that will prevent foundation related pressures from being transmitted to the final Railroad structure.

b) Influence Line

The need for underpinning an existing structure is determined by assuming an appropriate influence line. In general, this line originates at the invert of the new railroad structure and extends upward in the direction of the existing structure at an angle from the horizontal as indicated in Case I through Case IX exclusively. The slope of the influence line is generally a function of the in-situ water and soil conditions analyzed, considering the Contractor's methods of dewatering and temporary earth support. If an existing structure's foundation subgrade is either entirely or partially located above the influence line, it will generally require underpinning.

c) Appropriate Lower Soil Stratum

Support of underpinning loads must be developed, as a minimum, to a level below the influence line. However, if the soil stratum at this level is incapable of supporting the final foundation loads, the underpinning structure must be carried to an appropriate lower soil stratum that is not only undisturbed by the excavation but fully capable of supporting the foundation loads. The bearing capacity of the soil beneath the underpinning structure shall be determined using Article 11, Section C-26-1103.4 of Reference Standard RS-9.

d) Average Soil Conditions

An average soil condition will be said to exist if the in-situ soil, whether homogenous or stratified, consists of medium compact granular soils and/or stiff clays. The extent of compaction of the granular soil or the relative consistency of clay soils shall be determined from the blow counts on the sampler spoon indicated in the boring logs for the particular project. The blow count data shall be used in conjunction with the method outlined in Reference Standard RS-5, pp. 77-81 for granular soils, and Reference Standard RS-6, p. 7-1-6, Table 1-3 for clays.

e) Poor Soil Conditions

A poor soil condition will be said to exist if the in-situ soil consists predominantly of loose granular soil and/or medium to soft clay. The extent of compaction of the granular soil or the relative consistency of the clay soil shall be determined by the same method as for Average Soil Conditions outlined above.

f) Water Present

Water is considered to be present when the depth of water, as measured from invert, is greater than or equal to H/2; where H is the distance measured from subgrade to the top of structure. If the depth of water is less than H/2, it is not considered a factor in the determination of the slope of the influence line.

3. Soil Strata Subject to Compression Due to Construction

Any layer of organic or inorganic soil in which there is a reasonable probability of compression or consolidation due to adjacent construction operations such as dewatering, vibrations, etc. If this type of soil is prevalent, the underpinning must be designed to a depth such that it is bearing on soil which is not subject to the above condition.

4. Factors Determining Underpinning Requirements and Methods

a) Proximity of Excavation to Existing Structure

b) In-situ Soil Conditions

c) Ground Water Conditions and Control Techniques

d) Foundation of Existing Structure

e) Type of Earth Retaining Structure Used (i.e., soldier beams and timber sheeting, interlocking steel sheet piling, etc.)

f) Loads Carried by Existing Structure

g) Dimensions of Excavation

h) Construction Related Conditions (workmanship, sequence of operations, etc.).

i) Rock Quality

j) Dry Pack Min. 3" Between Existing Foundation and New Underpinning Pier

k) Pier to interlock to adjacent pier.

5. <u>Design Guidelines</u>

The typical influence line cases, pp. UP-7 to UP-15, are presented to provide general guidelines for determining the extent of underpinning and/or maintenance required. In order to fully utilize these guidelines, structures that may require underpinning are to be investigated to determine existing conditions. The available data concerning the type of foundation present, its physical condition, current use, previous underpinning, etc., is to be used in arriving at an appropriate underpinning scheme.

6. <u>Underpinning to Rock</u>

In general, when a building foundation rests on rock or the underpinning pier reaches rock, the underpinning problem is abated, assuming the rock is sound rock. However, if the rock is soft or disintegrated, underpinning must be carried through this to sound rock or to a suitable influence line. If the future excavation is to be made alongside the underpinning and deeper than the surface of the sound rock, a problem may arise if faults or slips exist in the rock so that a slide could easily take place. In such cases, the underpinning must be carried deeper to avoid the danger, or steps must be taken to prevent damage by adequate bracing against the rock face, or rock bolts installed into the rock face to prevent it from sliding. In any event, it is essential that the rock be carefully line drilled in the vicinity of the underpinning to help minimize the danger of overbreakage.

The quality of rock shall be determined by the "Percent Recovery Method" as outlined in Reference Standard RS-9, Article 11, Section C-26-1103.1, Classification of Soil Materials.

The Contractor should consider the use of RQD ("Rock Quality Designation"), as outlined in Reference Standard RS-10, as it pertains to indicating possible zones of weakened rock or sliding planes.

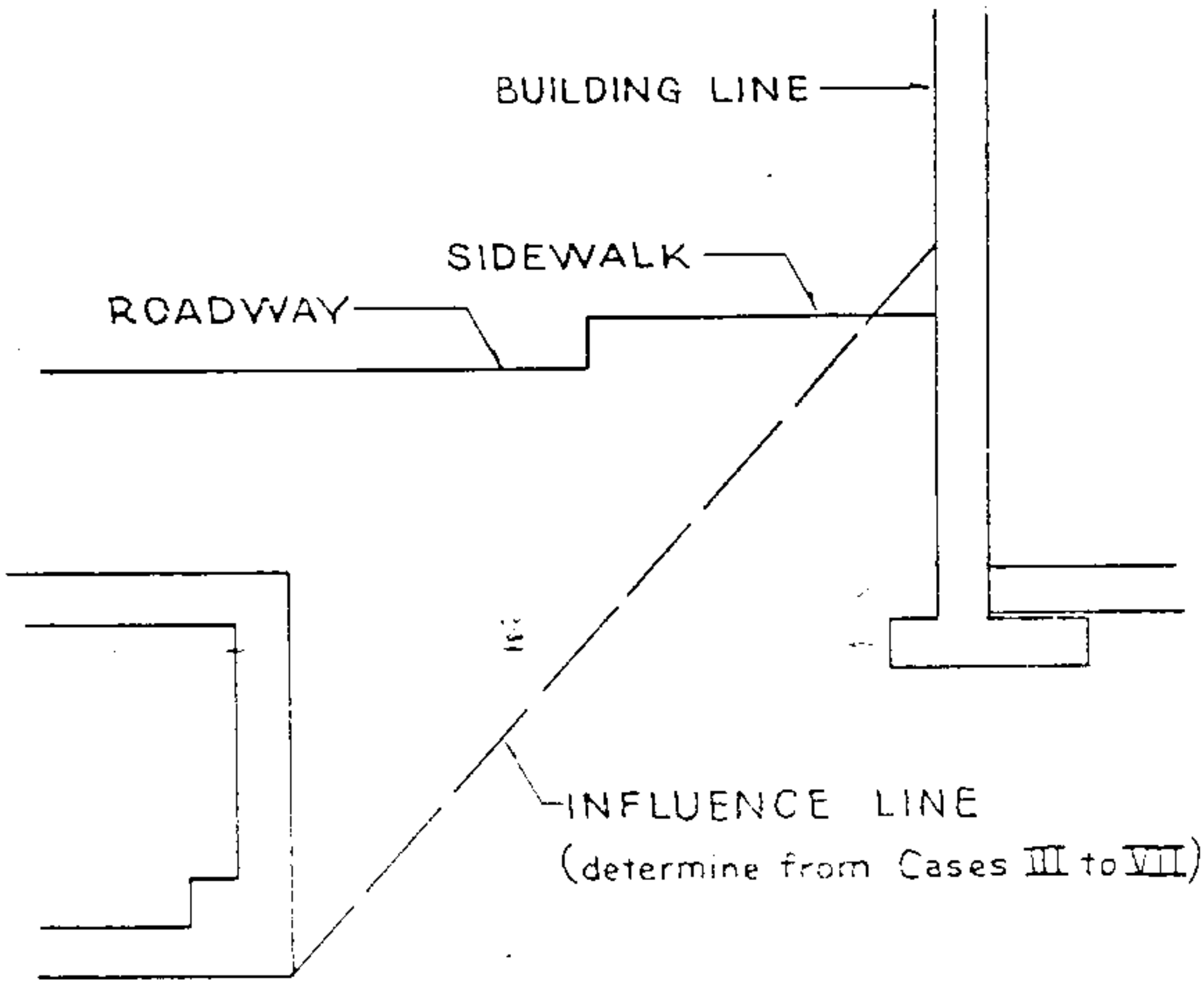
7. <u>Interior Columns</u>

Underpinning of interior columns may not be required if a continuous concrete underpinning wall or retaining wall is used to support or retain the loads from the wall of the building and which will prevent the loss of soil in the vicinity of the interior footings during excavation.

The requirement of underpinning interior columns, in the absence of a continuous underpinning wall, may be waived with the approval of the Engineer for light one and two story structures.

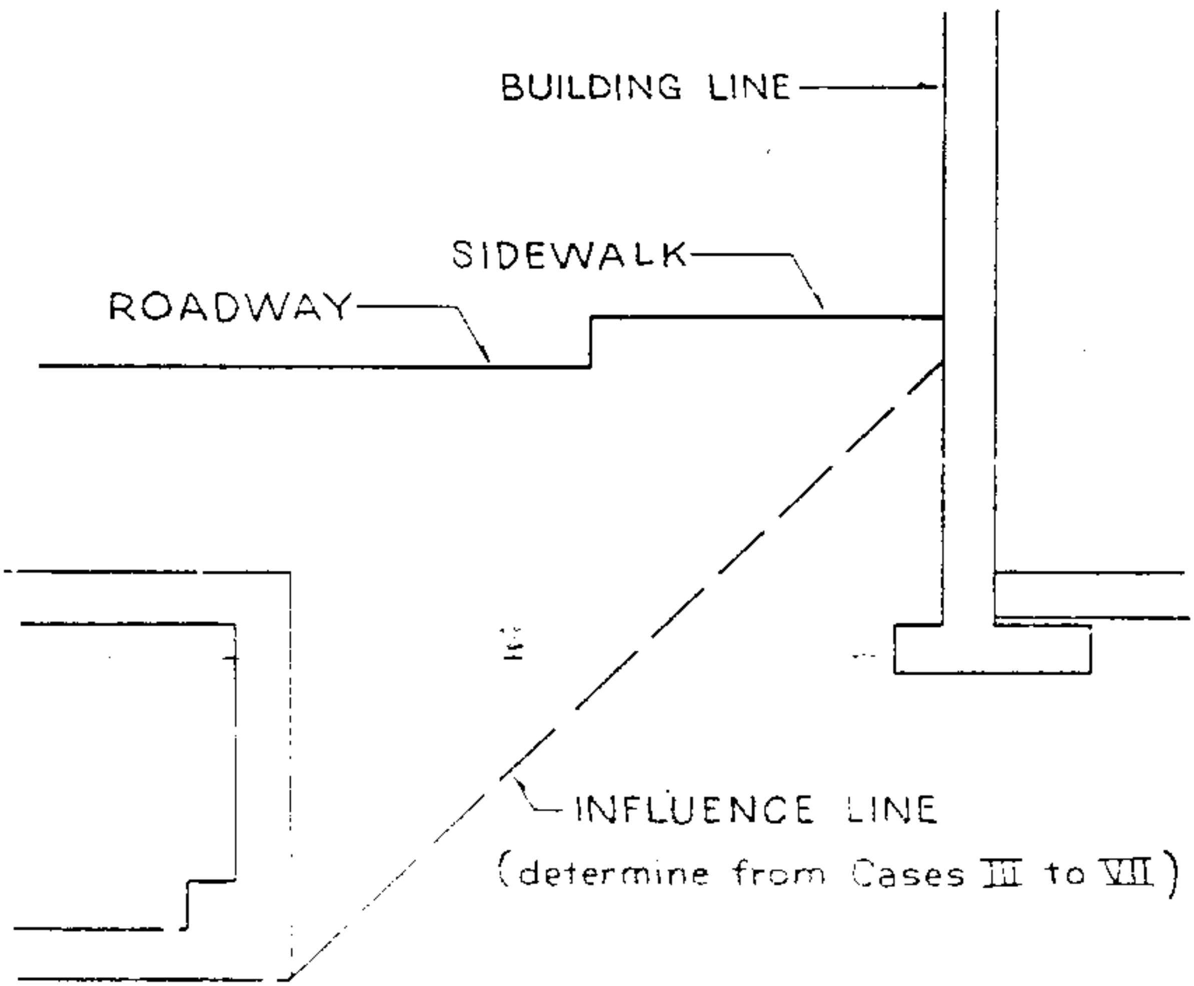
INFLUENCE LINES

CASE I No Underpinning or Maintenance Required
Influence line does not intersect building line below the ground surface.



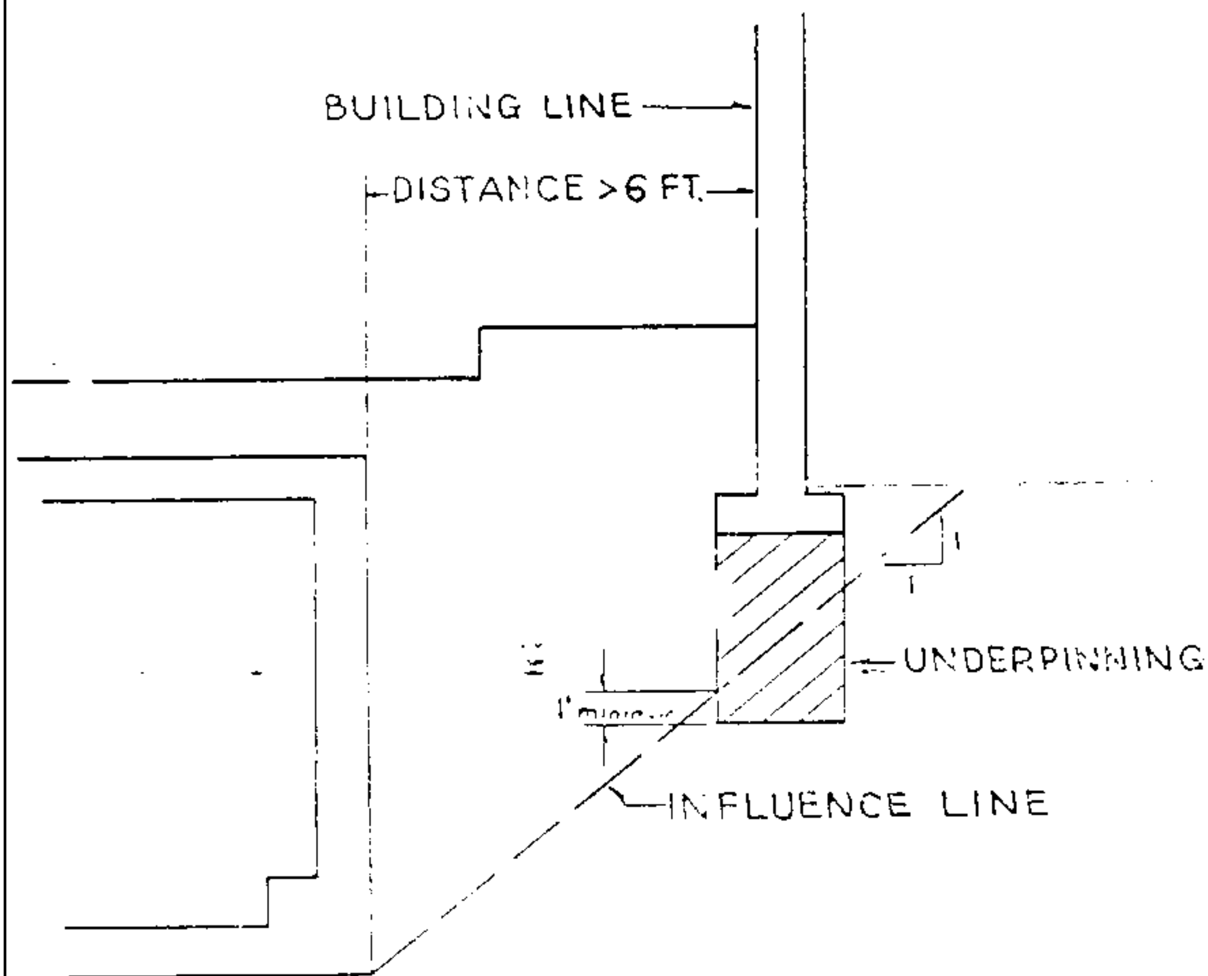
INFLUENCE LINES

CASE II   No Underpinning But Maintenance May Be Required



INFLUENCE LINES

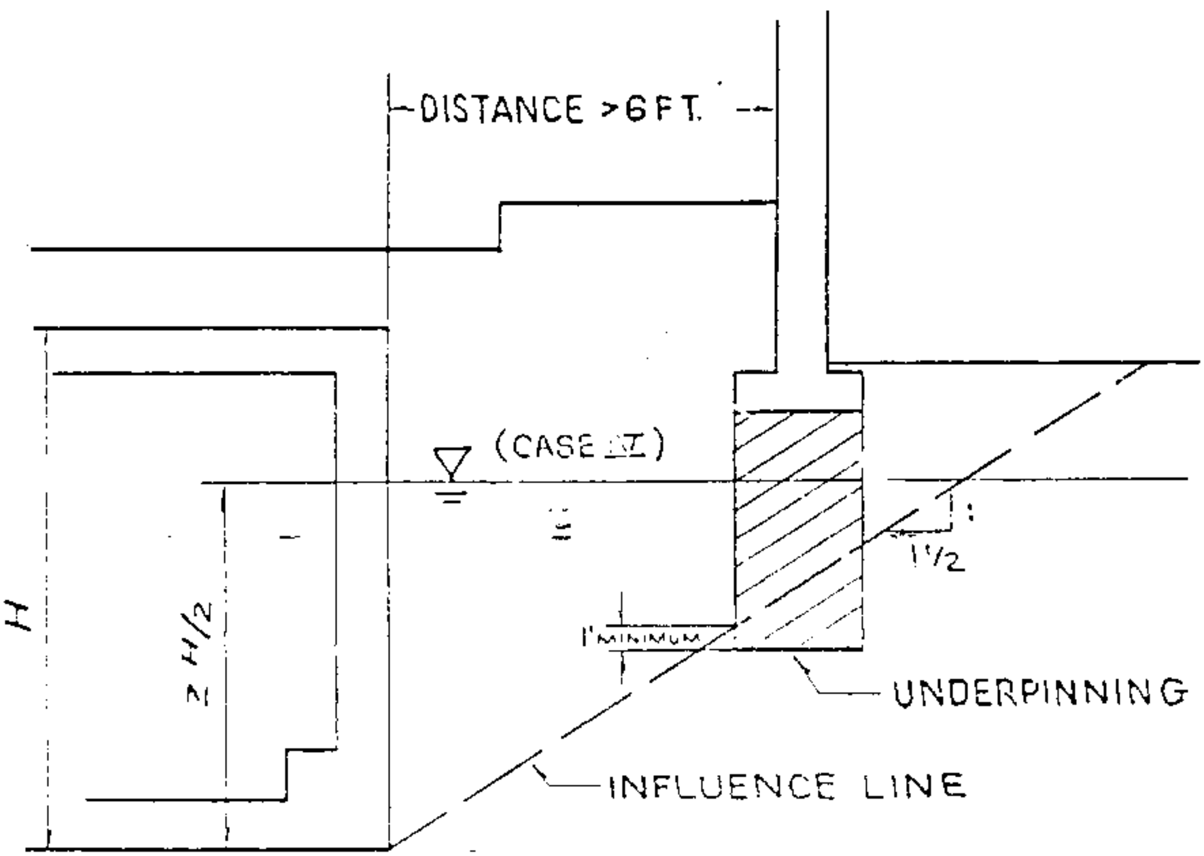
CASE III Average Soil Conditions
No Water (or below subgrade of excavation).
Building Lire More Than 6'-0" Distant.



INFLUENCE LINES

CASE IV Average Soil Conditions
Water Present

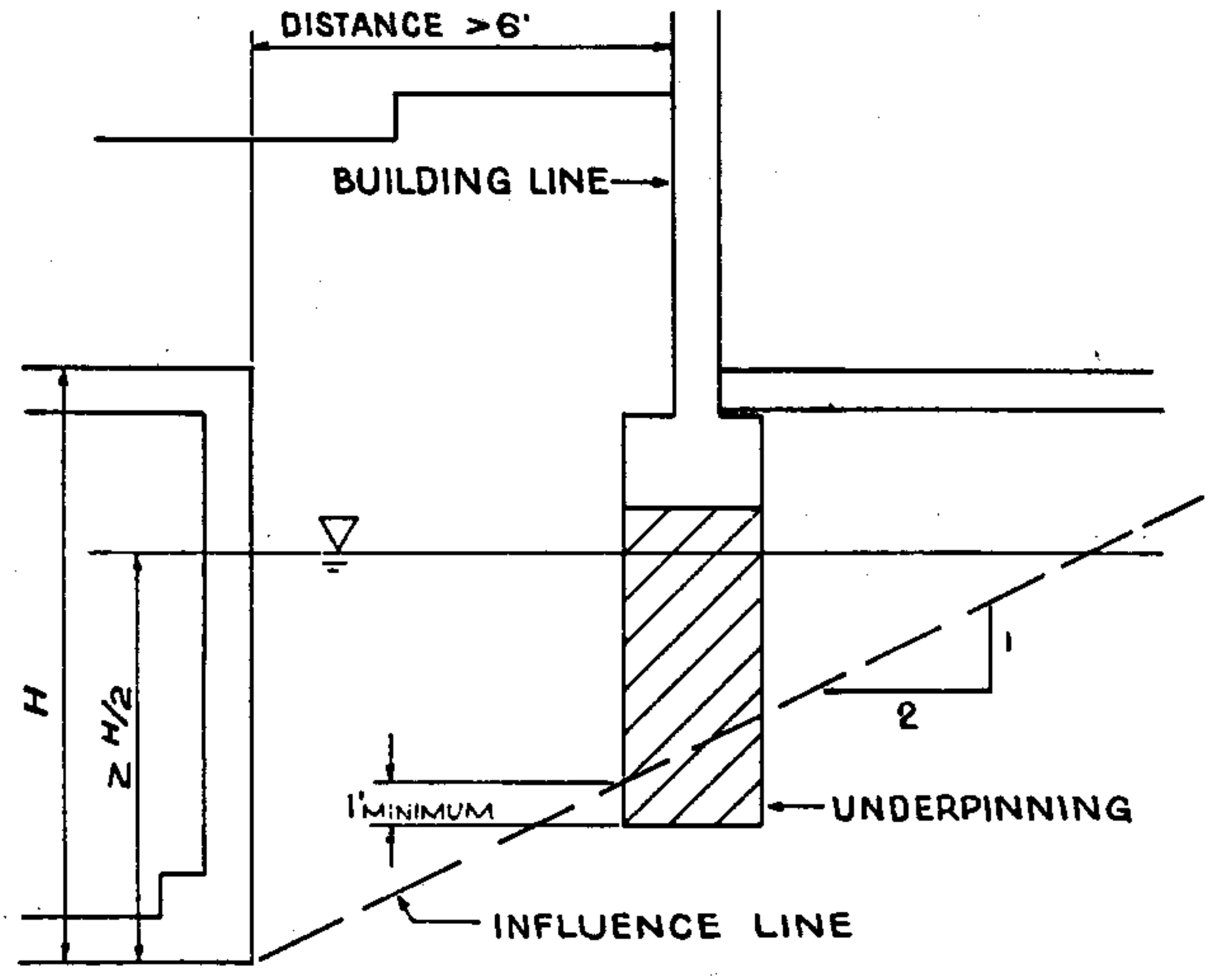
Case IVA Poor Soil conditions
No Water Present
Building Line More Than 6”-0” Distant
(both cases)



In CASE IV, an influence line of l:l may be used if the ground-water level outside the excavation is either maintained at pre-construction levels by the use of a water-tight earth retaining structure or effectively controlled by a dewatering technique approved by the Engineer.

INFLUENCE LINES

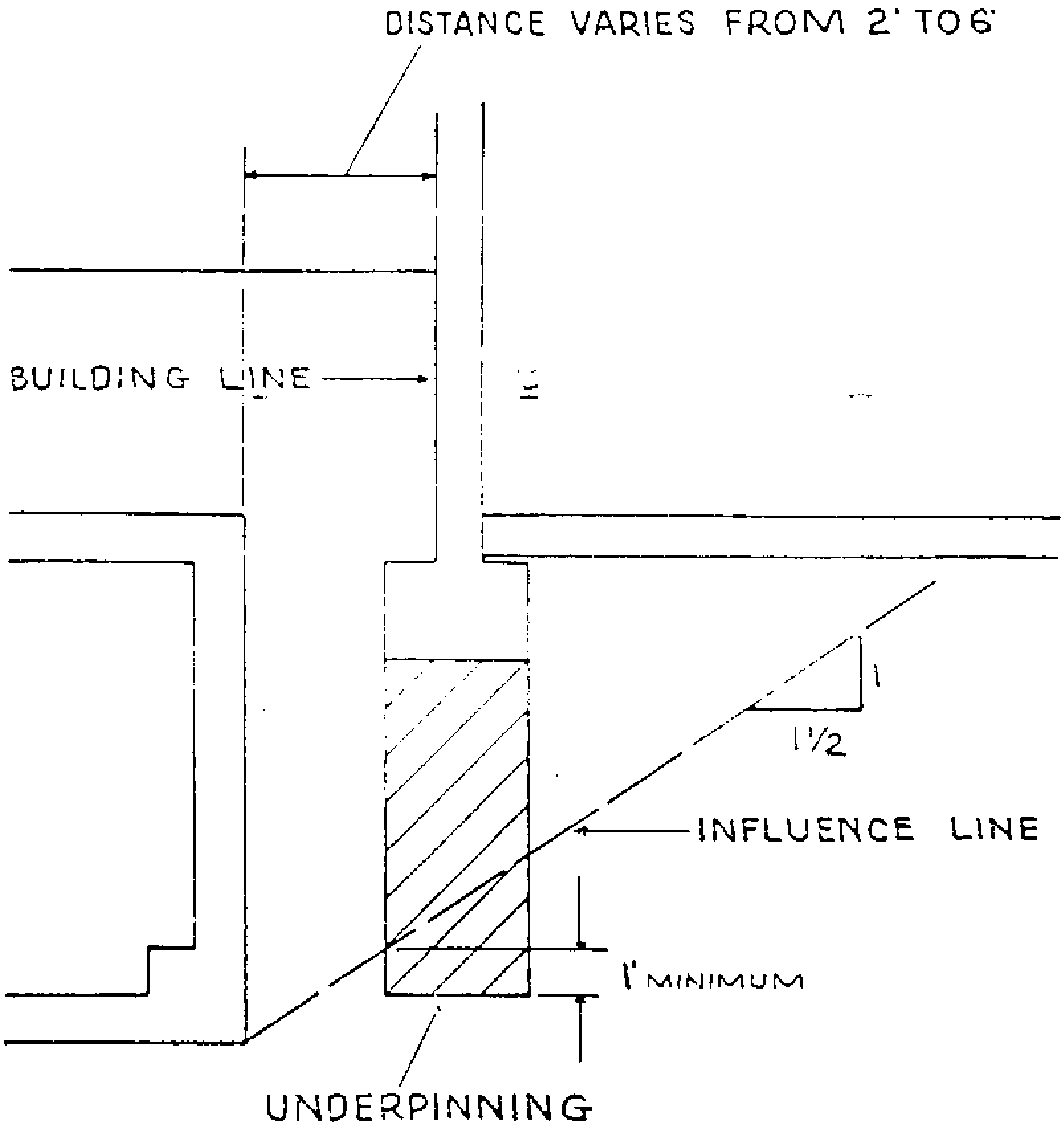
CASE V Poor Soil Conditions
Water Present
Building Line More Than 6'-0" Distant



An influence line of 1:1 ½ may be used if the ground-water level outside the excavation is either maintained at pre-construction levels by the use of a water-tight earth retaining structure or effectively controlled by a dewatering technique approved by the Engineer.

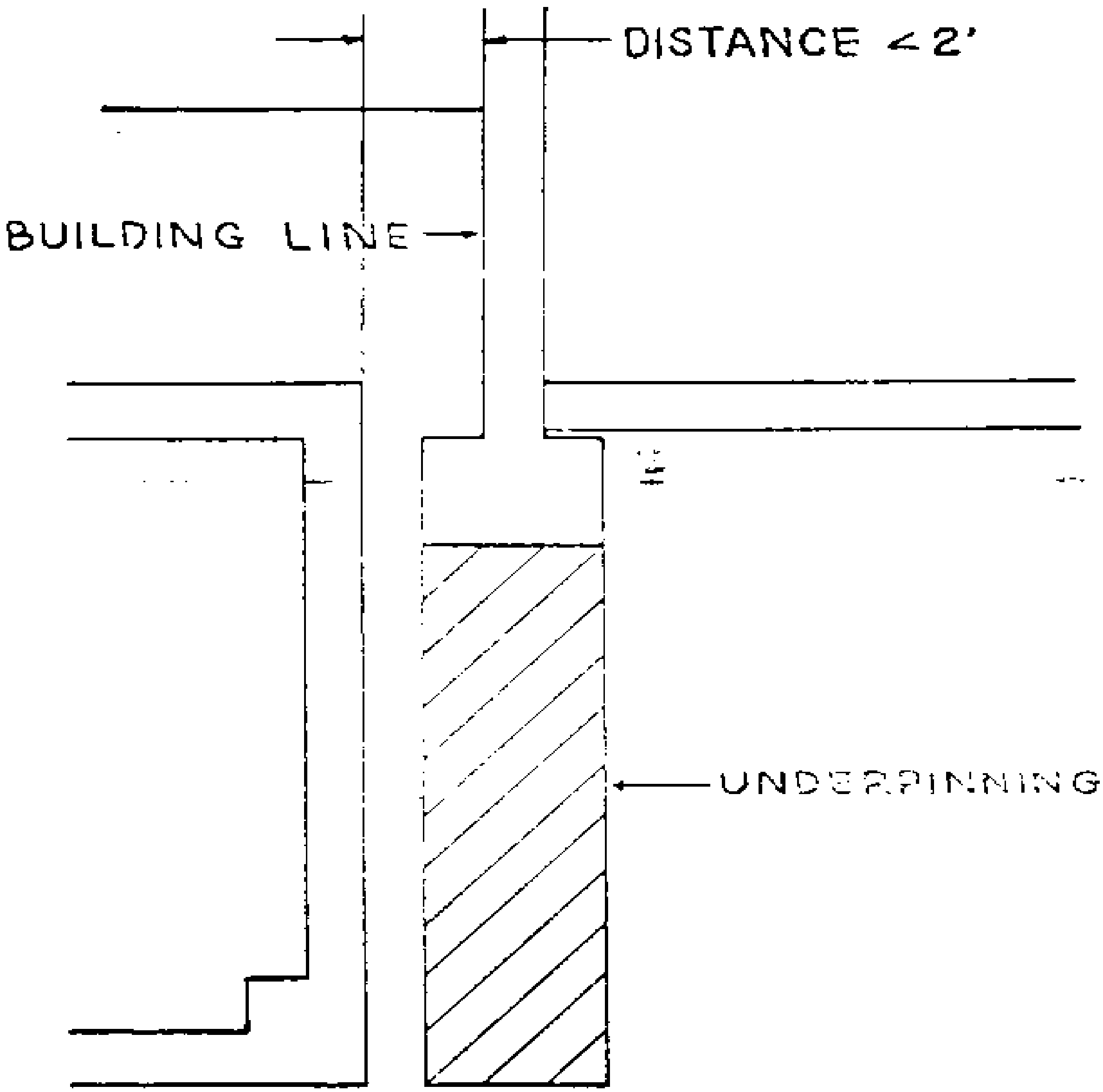
INFLUENCE LINES

CASE VI Average or Poor Soil Conditions
With or Without Ground-water
Building Line 2'-0" to 6'-0" Distant



INFLUENCE LINES

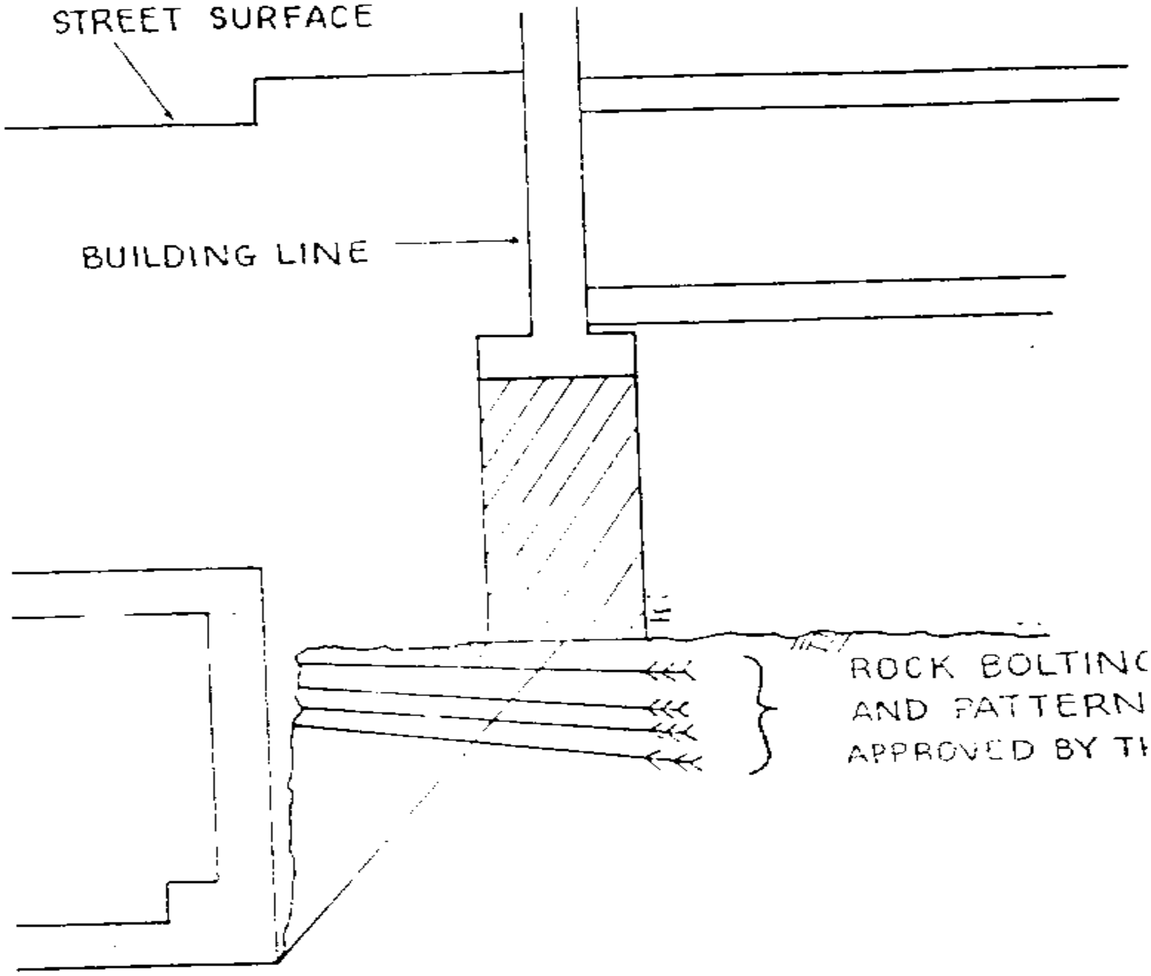
CASE VIII Average or Poor Soil Conditions
With or Without Ground-water
Building Line Less Than 2'-0" Distant



Carry underpinning to subgrade of new structure.

INFLUENCE LINES

CASE VIII Top of Rock Above Subgrade

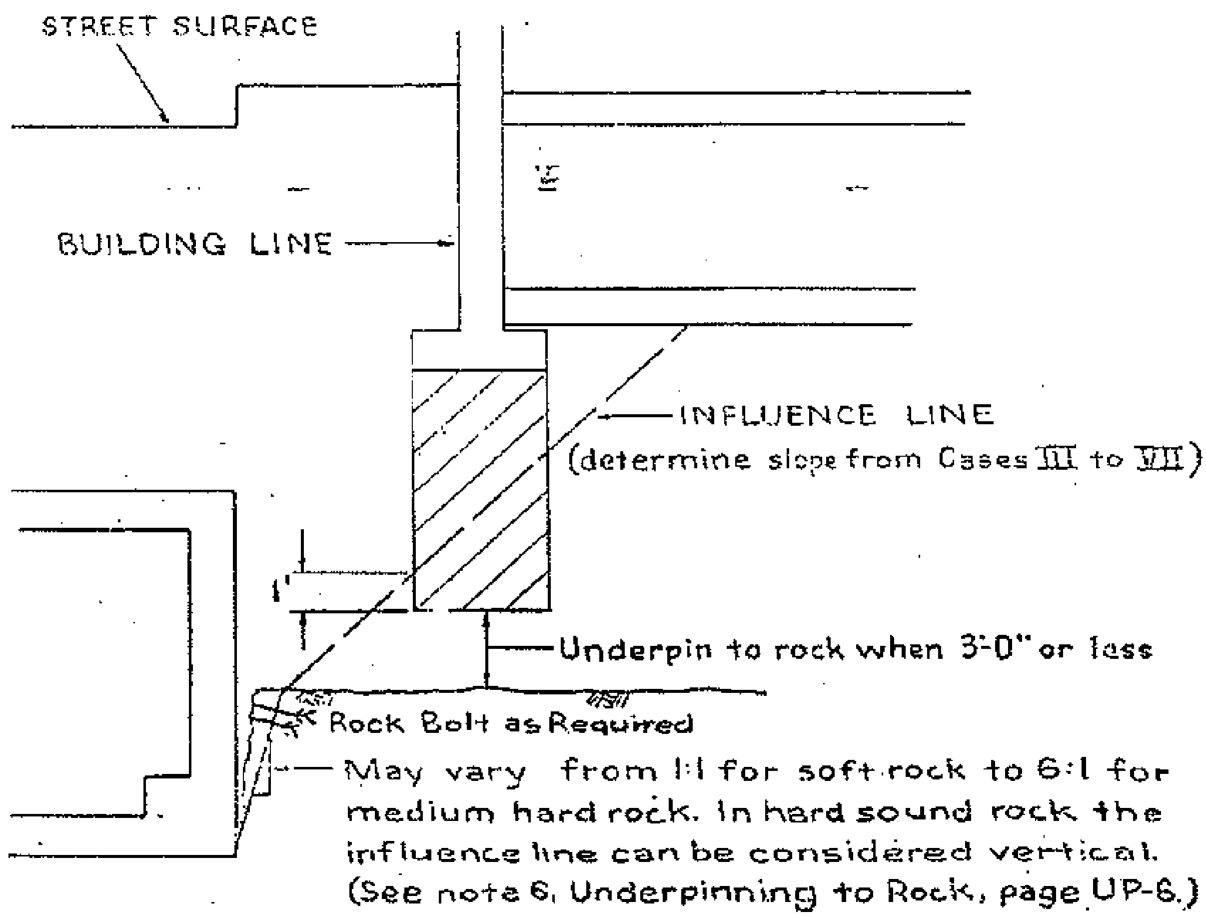


Underpinning should generally be carried to the top of sound rock or in its absence to an influence line which may vary from 1:1 for soft disintegrated rock to 6:1 for medium: hard rock.

Underpinning may be stopped at an elevation above the designated influence line if suitable reinforcement of the rock face adjacent to the underpinning can be provided as approved by the Engineer. (See Note 6, Underpinning to rock, page UP-6)

INFLUENCE LINES

CASE IX Top of Rock Above Subgrade



UNDERPINNING

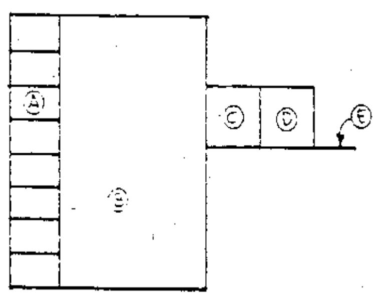
DESIGN EXAMPLE IV - UNDERPINNING ANALYSIS

DESIGN EXAMPLE IV
UNDERPINNING ANALYSIS

DETERMINE: FOR THE 5-STORY BRICK BUILDING (501 E. 78 ST.) ON BLOCK 1974 LOT 11, DETERMINE THE EXTENT, IF ANY, TO WHICH UNDERPINNING AND/OR MAINTENANCE MAY BE REQUIRED.

GIVEN: BORING LOGS B-4 AND B-5 AND BORING LOCATION PLAN (SEE PAGE UP-19).
FROM AN EXAMINATION OF BUILDING PLANS -
BUILDING FOUNDED ON CONTINUOUS FOOTING 10’ BELOW SIDEWALK ELEVATION.
DESIGN FOOTING LOAD 2 TSF.
INTERIOR COLUMNS 15’ O/C
NET LINE OF STRUCTURE 19.5’ FROM BUILDING LINE
INVERT OF STRUCTURE 35’ BELOW SIDEWALK.
HEIGHT OF STRUCTURE 20’
FOOTING WIDTH 3’

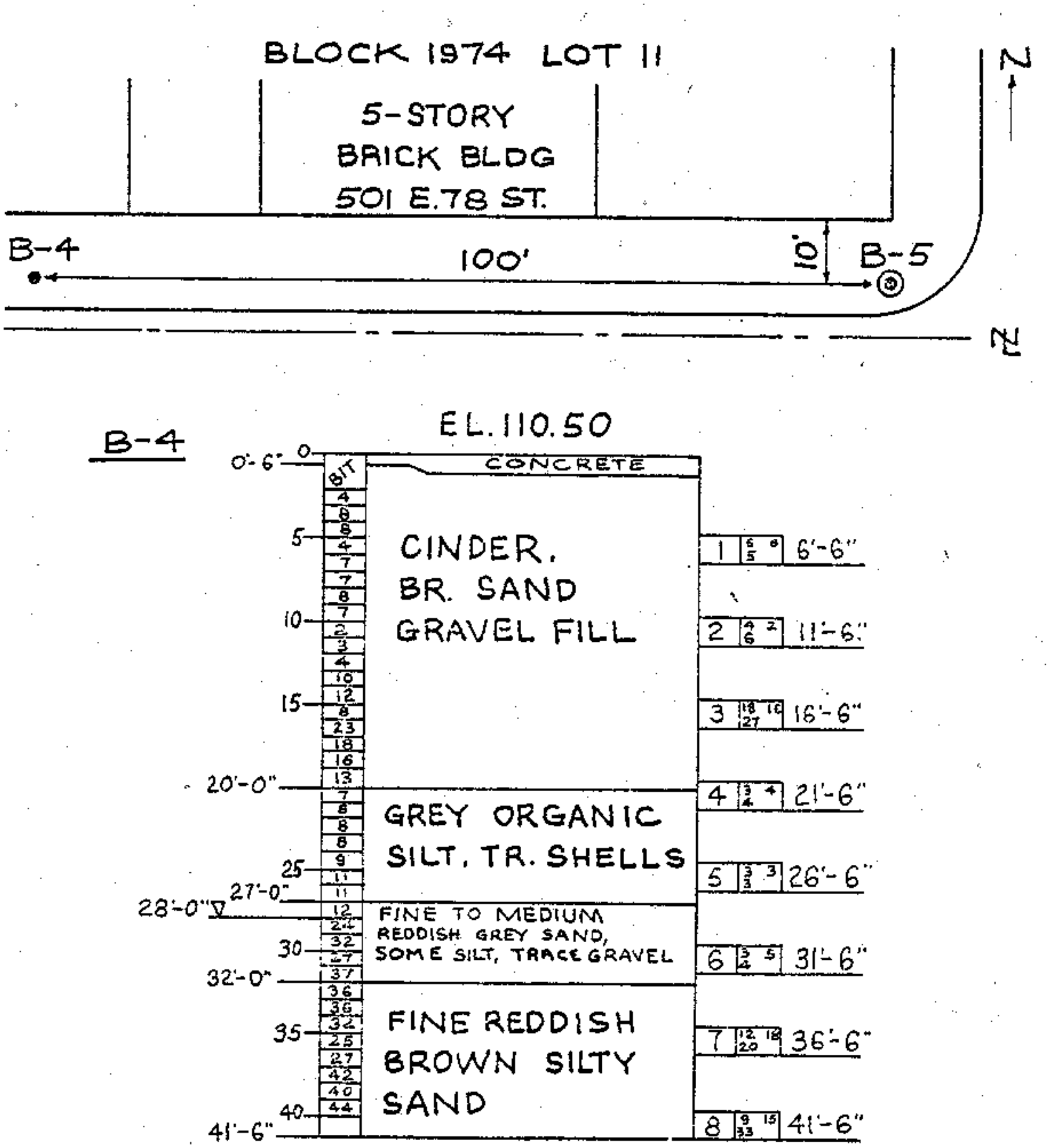
BORING LEGEND



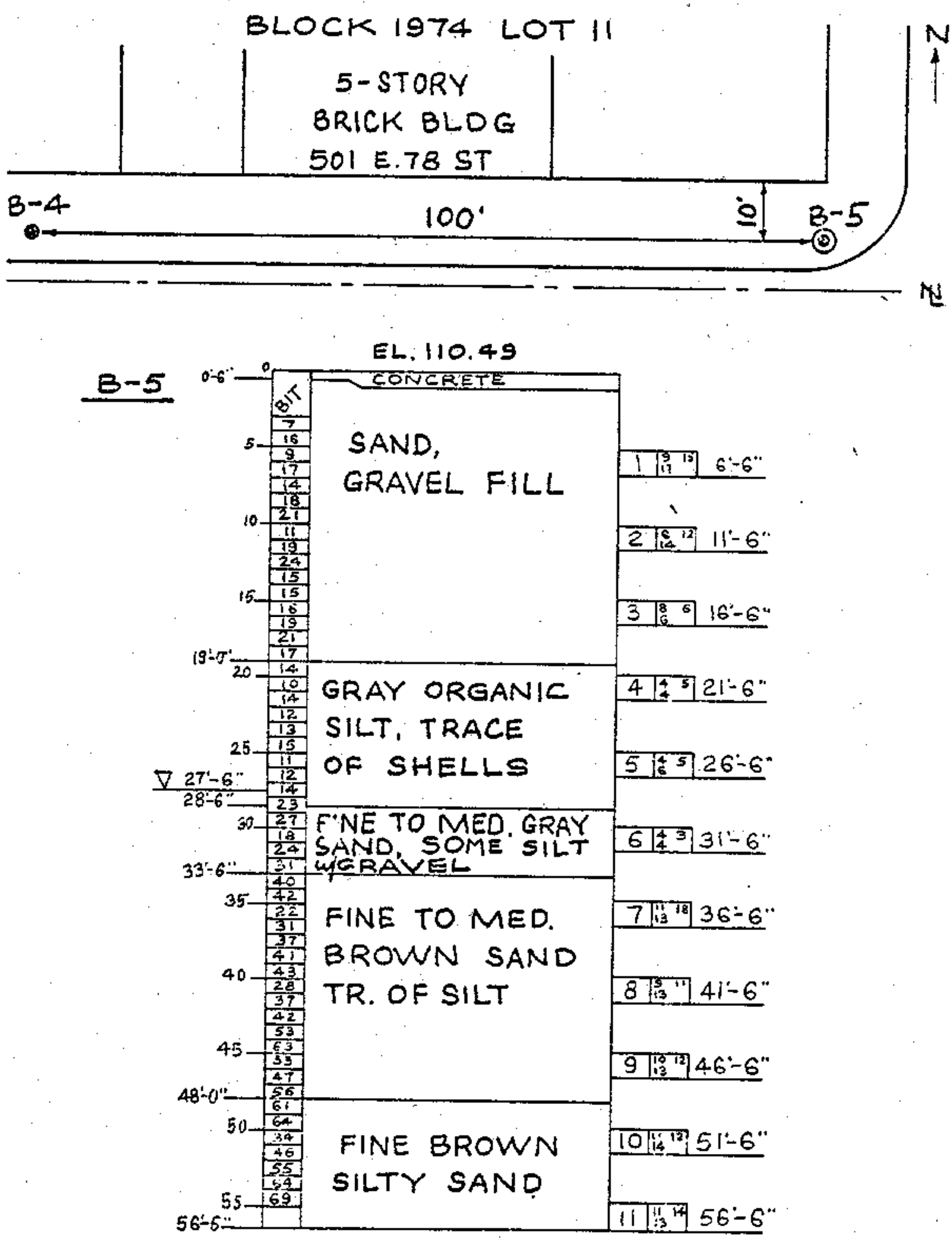
(A) N^{O} OF BLOWS OF A 300$^{\#}$ HAMMER FALLING 18” REQ’D TO DRIVE 2$^{1/2"}$ ϕ CASING 12”
(B) STRATA DESCRIPTION
(C) SAMPLE
(D) N^{O} OF BLOWS OF A 140$^{\#}$ HAMMER FALLING 30” REQ’D TO DRIVE 2” ϕ O.D.-1$^{3/8"}$ ϕ I.D. SPLIT SPOON SAMPLER 6”
(E) DEPTH AT END OF SAMPLE DRIVE

LOCATION OF BORINGS SHOWN THUS ⊗
LOCATION OF BORINGS WITH STRAINER HOLES SHOWN THUS ⊙

DESIGN EXAMPLE IV (CON'T)



DESIGN EXAMPLE IV (CON'T)



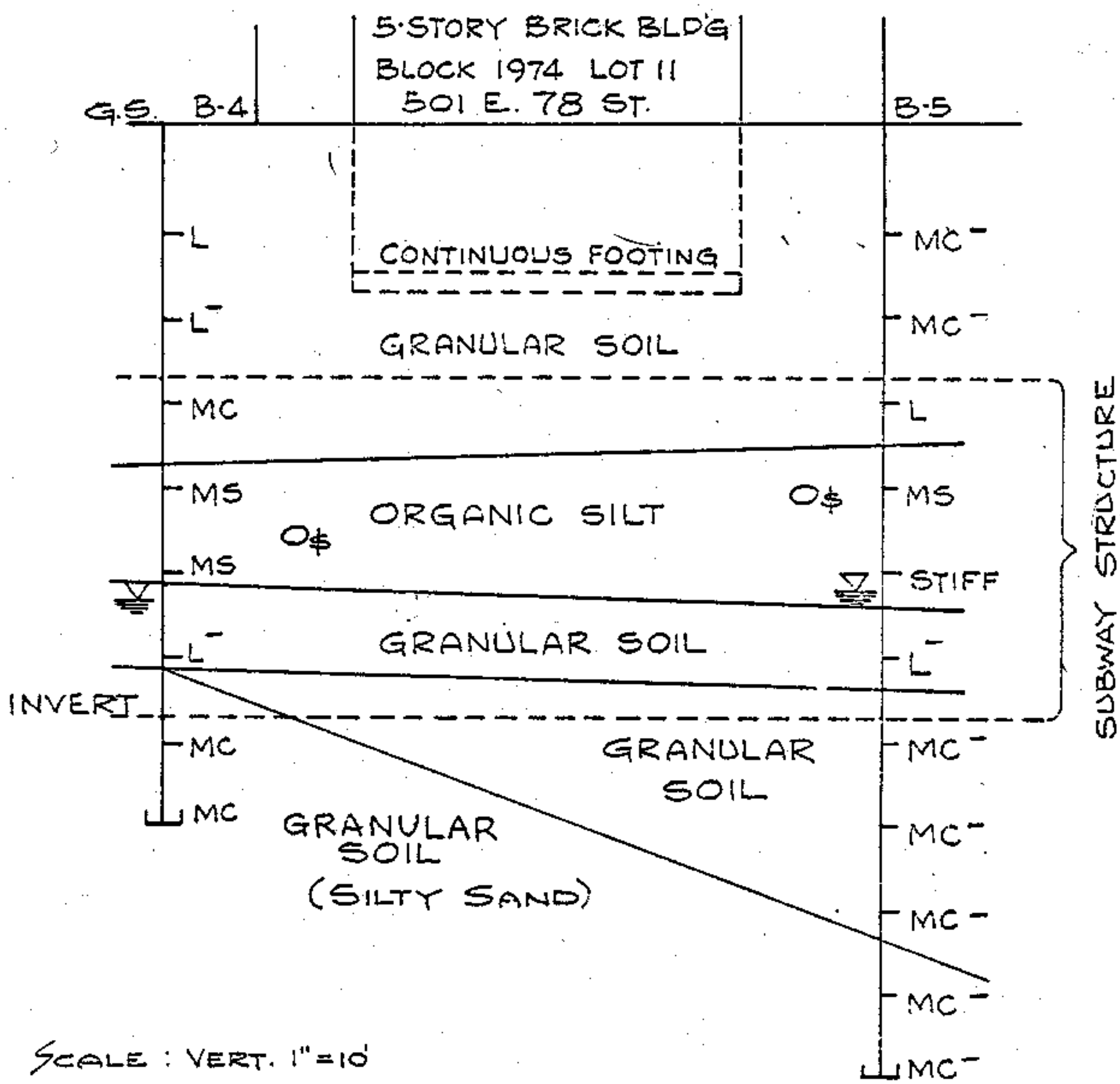
DESIGN EXAMPLE IV - (CON'T)

UNDERPINNING SOIL CRITERIA

AVERAGE SOILE: GRANULAR (NON-COHESIVE) → MEDIUM COMPACT or BETTER COHESIVE → STIFF or BETTER

POOR SOIL: GRANULAR (NON-COHESIVE) → RELATIVE DENSITY LESS THAN Mc COHESIVE → RELATIVE CONSISTENCY LESS THAN STIFF

WATER PRESENT: DEPTH OF WATER AS MEASURED FROM INVERT - H/2
H = DISTANCE FROM INVERT TO TOP OF STRUCTURE



DESIGN EXAMPLE IV - (CON'T)

INFLUENCE LINE DETERMINATION

1. THE BUILDING LINE IS 19.5' FROM THE STRUCTURAL NET LINE.

2. EXAMINATION OF THE BORING LOGS, IN CONJUNCTION WITH REFERENCE STANDARD RS-5 FOR GRANULAR SOILS AND REFERENCE STANDARD RS-6 FOR COHESIVE SOILS, RESULTS IN THE SOIL PROFILE SHOWN ON PAGE UP-21. BASED ON THIS PROFILE, INTERPRETED USING PARAGRAPH 2(d) AND PARAGRAPH 2(e) PAGE UP-4, A PREDOMINANTLY POOR SOIL CONDITION WITH RESPECT TO UNDERPINNING CAN BE EXPECTED.

3. THE LOCATION OF THE GROUNDWATER TABLE WITH RESPECT TO THE INVERT OF THE SUBWAY STRUCTURE IS 8'. (NOTE: LOCATION OF GWT BASED ON BORING B-5 BECAUSE THIS BORING IS A STRAINER HOLE) THIS IS LESS THAN $½^{H}$ CRITERIA FOR WATER PRESENT AS INDICATED IN PARAGRAPH 2(f) PAGE UP-4.

IN SUMMARY

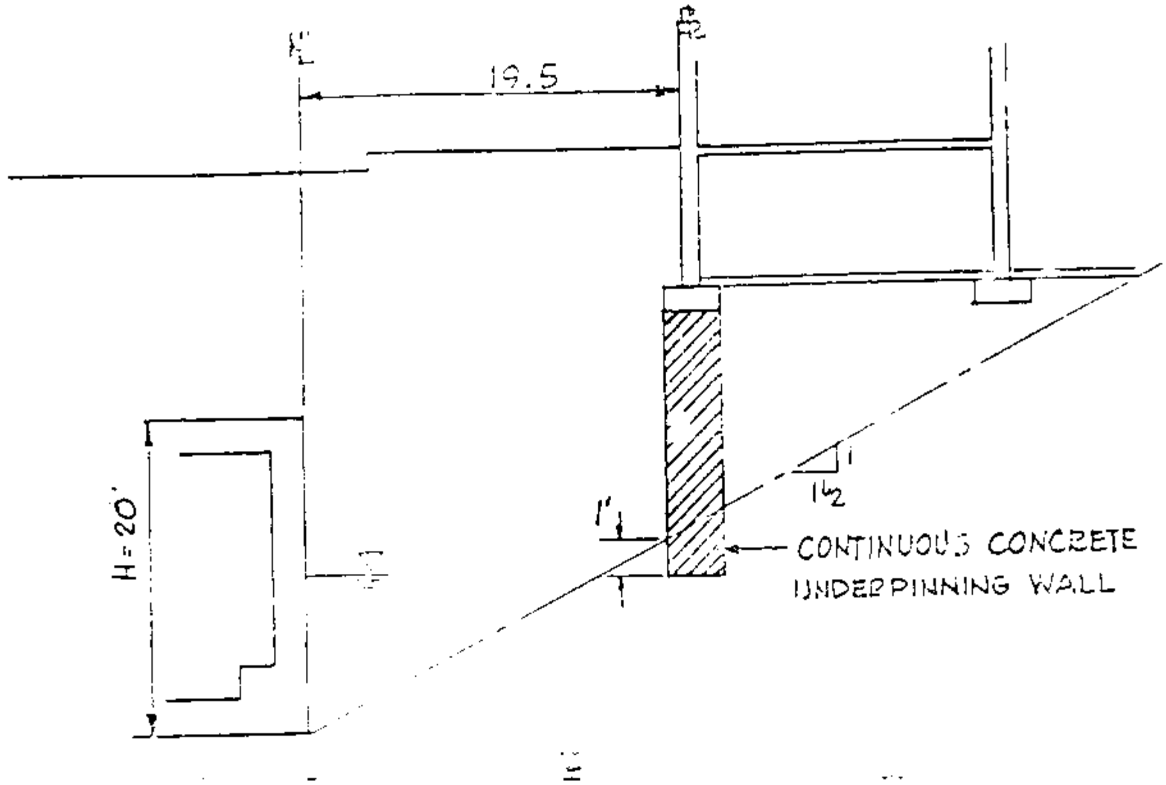
1 - THE BUILDING IS LOCATED GREATER THAN 6' FROM THE STRUCTURAL NET LINE.

2 - THE IN-SITU SOIL CONDITION IS PREDOMINANTLY POOR.

3 - NO WATER IS PRESENT THAT WILL AFFECT THE UNDERPINNING INFLUENCE LINE.

THESE FACTORS, TAKEN TOGETHER, INDICATE THE CASE IVA IS THE APPLICABLE CASE. THE RESULTING INFLUENCE LINE SLOPE FOR THIS CASE IS 1:1½.

DESIGN EXAMPLE IV (CON'T)\



UNDERPINNING TO BE CARRIED TO 1' MINIMUM BELOW INFLUENCE LINE AS INDICATED ABOVE. HOWEVER, AS SHOWN ON SOIL PROFILE PAGE UP-21 THIS WOULD PLACE THE BOTTOM OF THE UNDERPINNING WITHIN THE ORGANIC SILT STRATUM. ACCORDING TO PARAGRAPH 2(c), PAGE UP-3 UNDERPINNING MUST BE CARRIED TO A DEPTH SUCH THAT IT IS BEARING ON SOIL CAPABLE OF SUPPORTING THE FOUNDATION LOADS. THE POSSIBILITY OF EXCESSIVE SETTLEMENTS OF THE UNDERPINNING PIER IF FOUNDED IN THE ORGANIC SILT LAYER MUST BE CONSIDERED.

SINCE A CONTINUOUS CONCRETE UNDERPINNING WALL WILL BE USED UNDERPINNING OF THE INTERIOR COLUMNS WILL NOT BE REQUIRED AS PER PARAGRAPH 7, PAGE U-6.

EXTEND UNDERPINNING 1FT. INTO PREDOMINATELY FINE SAND (8-65) LAYER BELOW ORGANIC SILT.

BASED ON REFERENCE STANDARD RS-9, § C26-1103:4

NOTE (6), ALLOWABLE BEARING PRESSURE ON FINE SANDS:

BASIC ALLOWABLE BEARING PRESSURE = .1N

DESIGN EXAMPLE IV (Con't)

N WILL BE TAKEN AS THE LOWEST AVERAGE VALUE OF PENETRATION RESISTANCE WITHIN A DEPTH OF SOIL BELOW THE UNDERPINNING EQUAL TO ITS WIDTH (B = 3ft) = 3.67 BLOWS / 6IN. $\therefore$ N = 7.33 BLOWS / FT.

.1(7.33) =.733TSF   SAY 2 TSF(MIN)   SEE NOTE (C)

NO INCREASE DUE TO EMBEDMENT ALLOWED AS PER
NOTE (8)
NOTE(9) DOES NOT APPLY

<u>FOUNDATION LOAD:</u>

$$2\text{ TSF} + \frac{150\text{ pcf }(19\text{ft.})}{2000\text{lb / ton}} = 2 + 1.43 = 3.43\text{TSF}$$

<u>ALLOWABLE BEARING PRESSURE:</u>

$$2\text{ TSF} + \frac{80\text{ pcf }(19\text{ft.})}{2000\text{lb / ton}} = 2.76\text{TSF}$$

NOTE: ASSUME AVERAGE SOIL WEIGHT = 80pcf

AS PER § C26 - 103.4 $q\text{ allow} + Po \geq q\text{ found}$

WHERE q_{allow} = ALLOWABLE BEARING PRESSURE AS DETERMINED FROM TABLE 11-2 AND SUPPLEMENTARY NOTES

Po = STABLIZED OVERBURDEN PRESSURE

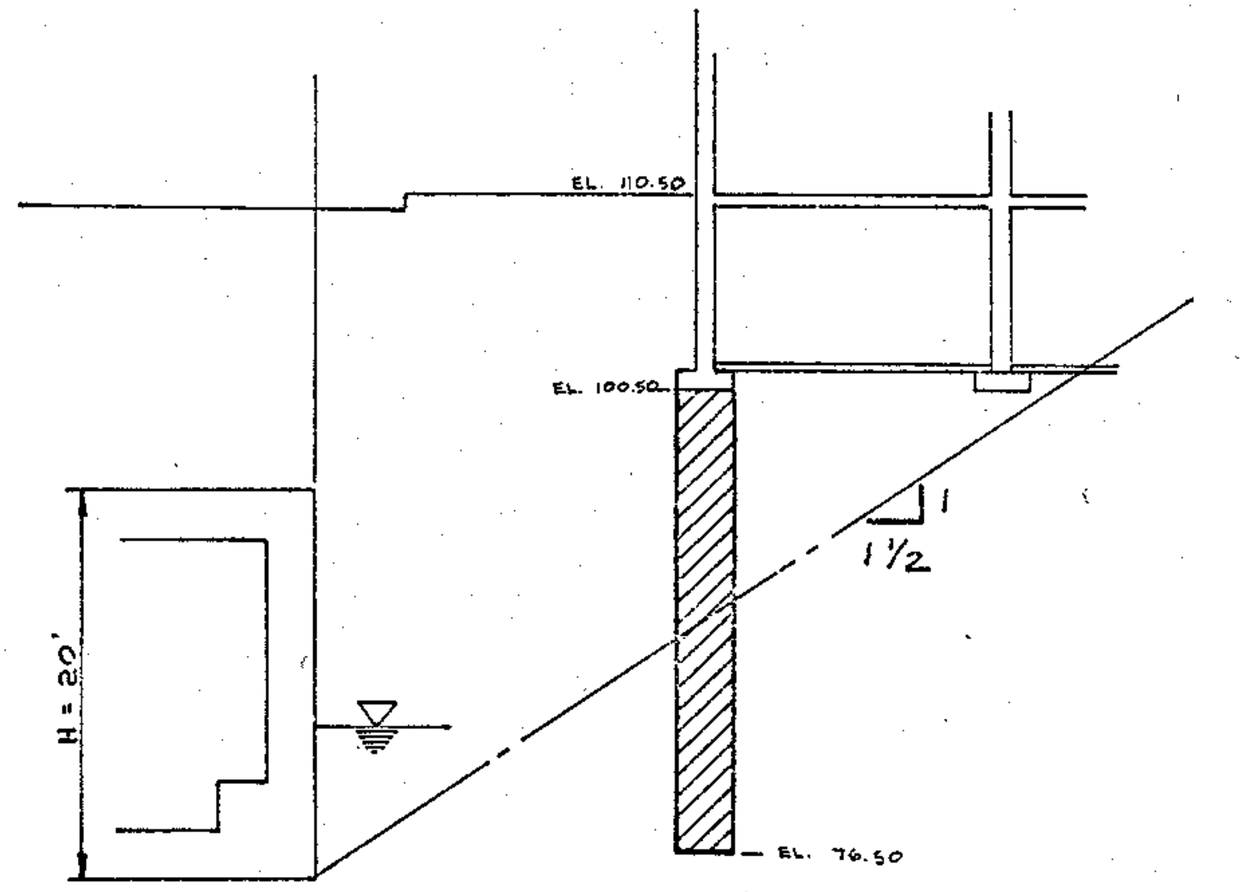
q_{found} = FOUNDATION LOAD

2.76 TSF < 3.43 TSF N.G.

THEREFORE, EXTEND UNDERPINNING 1ft. INTO PREDOMINANTLY SAND (7-65) LAYER BELOW FINE SAND LAYER.
BASED ON REFERENCE STANDARD RS-9, §C26-1103.4
NOTE (5) ALLOWABLE BEARING PRESSURE ON SANDS:

$$\frac{11+18+13}{3} = 14\text{BLOWS / 6 IN.} \quad \therefore \quad N = 28\text{ BLOWS / FT.}$$

DESIGN EXAMPLE IV (CON'T)



1(N) =.1(28) = 2.8TSF SAY 3 TSF (MIN)
SEE NOTE (5)

NO INCREASE DUE TO EMBEDMENT ALLOWED
AS PER NOTE (8)

NOTE (9) DOES NOT APPLY

FOUNDATION LOAD: $2\text{ TSF} + \dfrac{150(24)}{2000} = 3.8\text{TSF}$

ALLOWABLE BEARING PRESSURE:

$$3\text{ TSF} + \frac{80(24)}{2000} = 3.96\text{TSF}$$

3.96 TSF > 3.8 TSF O.K.

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SECTION TB

TIEBACKS

TIEBACKS SYSTEM

INTRODUCTION

When open cut excavations have earth support systems (e.g., Soldier Beams and Lagging or Steel Sheeting), which are not cantilevered at the invert of the cut, then the vertical soldiers or steel sheeting must be supported by either an internal bracing system or an external tieback system.

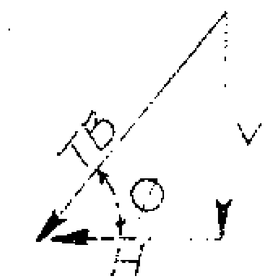
Tieback systems may be used in cuts greater than twenty feet wide, while horizontal bracing system are not practical for cuts greater than sixty feet wide.

The earth support system's loading impacts both axial compressive and dead load bending stresses on the internal bracing system's heavy wide flange steel section braces. These braces are within the construction area and may interfere with the placement of the new structure.

The earth support system's loading impacts axial tension stresses on the external bracing system's tendon tiebacks, either wire cables or solid steel anchor bars, which are anchored in the soil or rock medium which are located beyond the slip plane. These tiebacks are located beyond the excavated area and beyond the construction of new structure.

Earth tiebacks are usually placed 15' below the horizontal as to limit the vertical component of the tiebacks which must be resisted by the soldier beam or steel sheeting. The variation of the vertical and horizontal components as the angle ϕ varies from 7.2 to 48.6 is shown on Table A.

TABLE A



ϕ	TB	H	V
7.2^o	1	0.99	0.13
15^o	1	0.97	0.25
22^o	1	0.93	0.325
30^o	1	0.87	0.5
48.6^o	1	0.44	0.75

The earth tieback system has been used on the Authority's Program contracts for the construction of an underground (below stress surfaces) sub-station located at Park Row, and the support of a 2 feet high embankment supporting an operating railroad at Archer Ave.

The rock tieback system has been used on the Authority's Program Contracts for the construction of large shafts found in rock. These shafts were constructed as part of the new route program. The approximate size of these shafts is (60' x 100' wide x 120' deep).

TIEBACKS

GENERAL NOTES

1. The maximum test load shall be equal to 120 percent of design load.

2. The lock off load shall be equal to 80 percent of design load.

3. Before stressing of earth anchors, grout to attain a minimum compressive stress of 3000 psi.

4. Grout strength shall be checked by means of 2" grout cubes, tested at certified testing laboratory and the laboratory report on test performed shall be furnished to the Authority's Engineer.

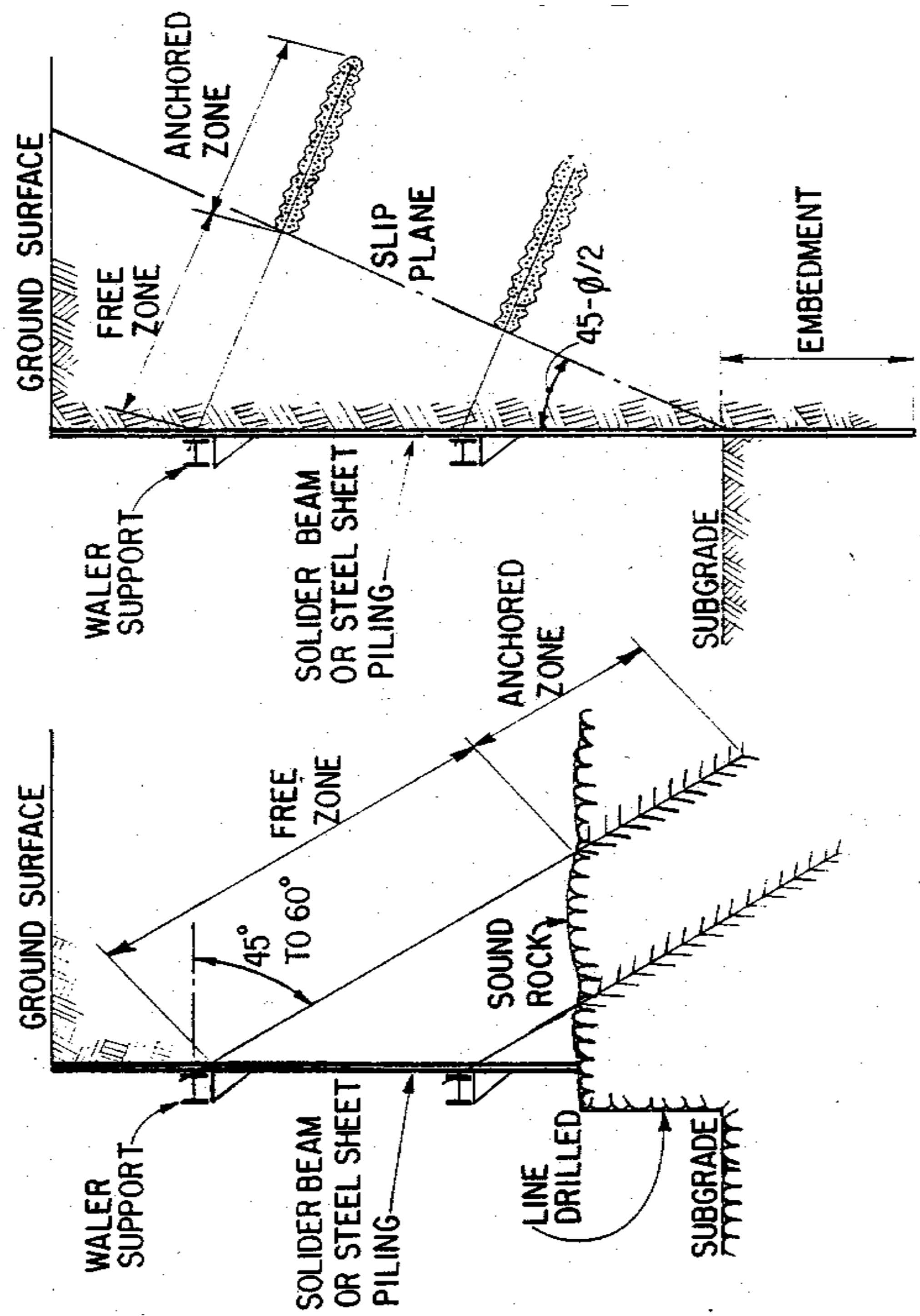
5. The minimum safety factor for tendons of tiebacks is 2.0.

6. The medium which the tendon is anchored to shall be:

   . Cementatious Grout - in granular soil.

   . Epoxy Grout - in class 65-3 or better rock.

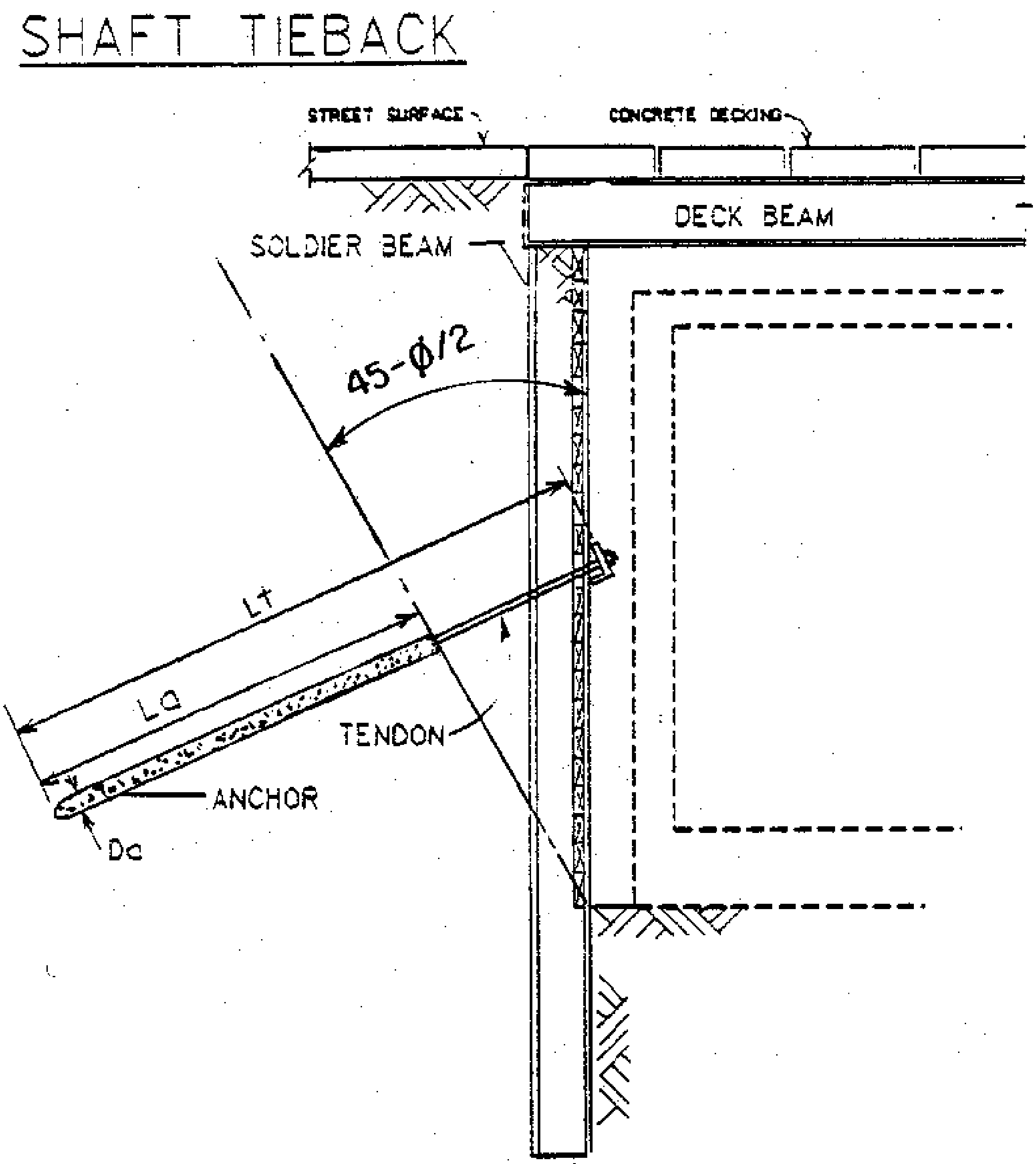
TIEBACKS



EARTH ANCHORED TIEBACK

ROCK ANCHORED TIEBACK

TIEBACKS



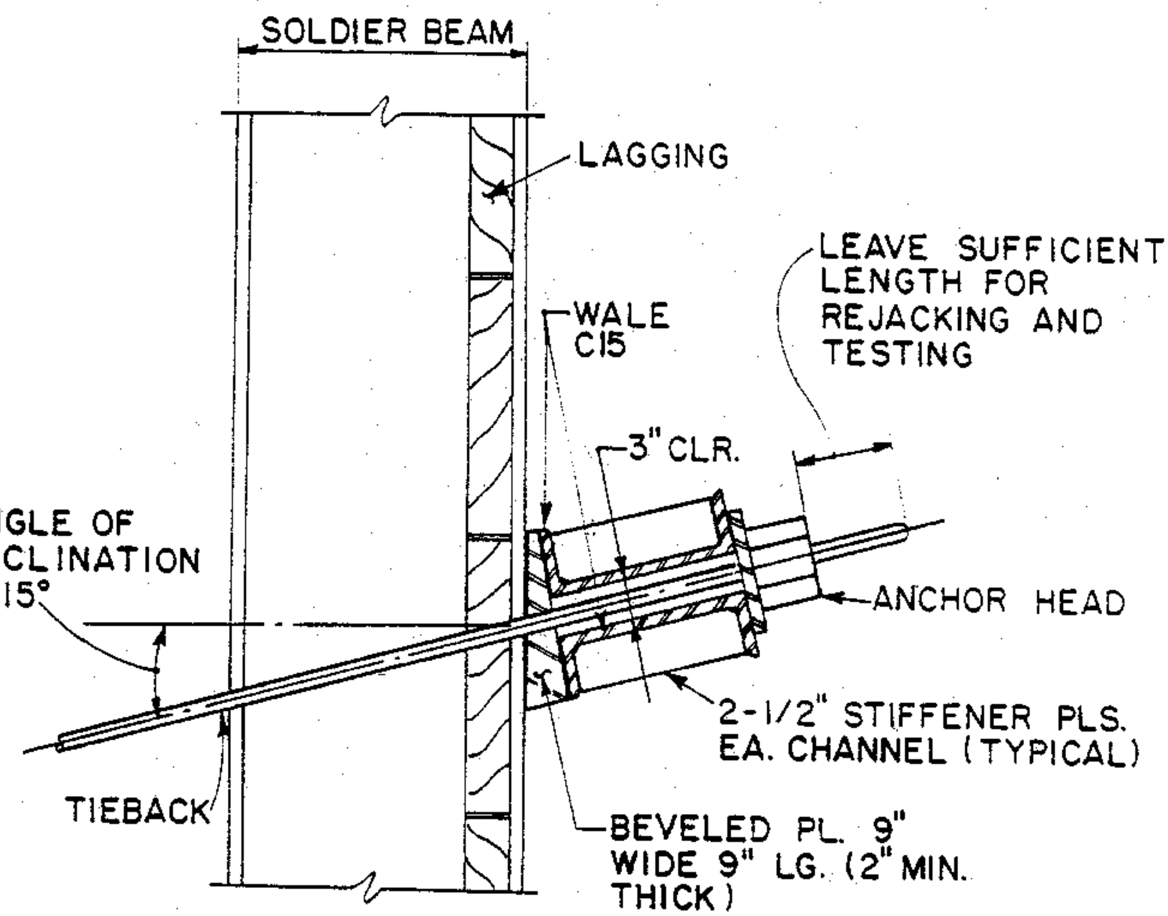
P_S = Ac x fg = πDaLaFg

- P_S = Anchor capacity
- Ac = Cylindrical area of soil-anchor bond
- Fg = bond stress*
- Da = Diameter of the anchor
- La = Length of the anchor

* See Table T-1 & T-2

TIEBACKS

EXAMPLE OF JACKING DETAIL



TENDON TIEBACK

- MULTI STRAND CABLE
- SOLID DYWIDAG BAR

TIEBACKS

BOND STRESSES

TABLE T-1 TYPICAL VALUES FOR UNIT SOIL - ANCHOR BOND STRESS

SOIL TYPE	STANDARD PENETRATION RESISTANCES (BLOWS FT)	BOND STRESS BETWEEN ANCHOR AND SOIL (KIPS/FT^2)
SANDY CLAY	3 - 6	0.50 - 1.00
MEDIUM CLAY	4 - 8	0.75 - 1.25
FIRM CLAY OR STIFFER	OVER 8	1.00 - 1.50

TABLE T-2 TYPICAL VALUES FOR UNIT ROCK - ANCHOR BOND STRESS

ROCK TYPE	BOND STRESS BETWEEN GROUT AND ROCK (LBS/FT^2)
SANDSTONE	120 - 250
SOFT SHALES	30 - 120
SLATE AND HARD SHALES	100 - 200
SOFT LIMESTONE	150 - 220
HARD LIMESTONE	300 - 400
GRANITE & BASALT	250 - 800

From Reference Standard RS-12

SECTION UT

UTILITIES SUPPORT

SUPPORT & PROTECTION OF UTILITY COMPANIES' AND CITY AGENCY CABLES DURING CONTRACT CONSTRUCTION PHASE

1.0 POLICY

1.1 To control the manner in which a Utility Company or a City Agency cable plant is exposed, supported and protected during the construction phase of a Transit Authority construction contract.

2.0 PURPOSE

2.1 To establish guidelines for Transit Authority and Consultant field personnel to implement the above policy.

3.0 SCOPE

3.1 This guideline applies to all Utility Companies' and City Agency cable plants in the City of N.Y., mapped streets, private areas, and Transit Authority property.

4.0 DEFINITIONS

4.1 Utility Companies or City Agency

a. Consolidated Edison Co. of New York
b. New York Telephone Co.
c. Empire City Subway Co., Ltd.
d. Western Union Telegraph Co.
e. Cable T.V. Company
f. N.Y.C. Fire Department
g. N.Y.C. Traffic Department

4.2 Utility Companies' and City Agency Cable Plant

4.2.1 Types of Cable

a. Single conductors, multiple conductors, for high tension, low tension and customer services - used by Con Edison Co.

b. Con Edison High Pressure Pipe Cable Transmission Lines (Oil - O - Static Pipes) consist of one or more continuously welded steel pipe lines, factory coated, with field coating of the welded joints to prevent corrosion. Each pipe line contains three (3) single conductor cables surrounded by a pressurized di-electric oil. The voltages range from 38,000 to 345,000 volts.

c. Multiple wire communication cable is used by N.Y. Telephone & Western Union Telegraph Co.

d. Other cables - single or multiple cables - used for fire alarm, traffic or Cable TV systems.

4.2.2 <u>Types of Conduits And Ducts</u> (Used for installation and protection of underground cables between manholes or service boxes, or to customers).

a. Single or multiple vitrified clay tile ducts.

b. Single or multiple precast concrete ducts.

c. Single fiber conduits.

d. Single P.V.C. (Plastic) conduits.

e. Single wooden ducts.

f. Single steel pipe conduits.

g. Single fiberglass conduits.

4.2.3 <u>Manholes, Service Boxes & Vaults</u>

a. <u>Manholes</u> - are large underground splicing chambers housing a large quantity of high tension, low tension electric cables, communication or telegraph cables, with access from the street surface through manhole covers or removable gratings (power cables & communication cables do not jointly occupy the same manhole).

b. <u>Service Boxes</u> - are small underground splicing chambers housing a small quantity of low tension or services consisting of electric, communication or telegraph cables, with access from the street surface through service box covers or removable gratings (power cables & communication cables do not jointly occupy the same service box).

c. <u>Vaults</u> - are underground structures to house transformers with access from the street surface through removable gratings to facilitate installation or replacement of transformers (incoming cable is high tension, out going cables are low tension).

d. <u>Type of Construction</u>
- field poured, reinforced concrete
- precast, reinforced concrete
  - brick

5.0 GUIDELINES & PROCEDURE

5.1 Prior to excavation, the Contractor notifies all Utility Companies and City Agencies, to mark out the street, or by other mutual means between them, to indicate the locations of their subsurface structures in compliance with N.Y. State Department of Labor, Industrial Code Rule 53.

5.2 The field engineer or inspector should coordinate with the Utility Co. or City Agency involved when the Contractor is performing work on underground facilities, to provide notice and to obtain assistance as required.

5.3 After the street pavement is removed, the Contractor excavates pits using hand-held tools utilizing only human power, below any subsurface structure, prior to driving soldier beams or piles.

5.4 The Contractor shall not employ powered or mechanical excavating equipment closer than 4 inches in any direction from a known underground facility. An exception to the above is when removing the concrete or masonry encasement from a conduit or duct line; then a hand-held tool not exceeding the power of a pneumatic chipping gun with a one and one-eight inch (1-1/8") diameter piston and a two inch (2") stroke, operated at 110 P.S.I. air supply may be used, provided the procedure is demonstrated by the Contractor to the Engineer, and approved.

5.5 The Contractor must install shield between cables & interior walls and roof of manholes and service boxes before demolition of such structures.

5.6 The Contractor shall install temporary wooden manholes or service boxes, providing the same facilities for racking and supporting the cables, to replace the structures removed in Paragraph 5.5. The temporary manholes and service boxes shall be constructed of fire retardant pressure impregnated wood consisting of two inch (2") tongued and grooved lumber or 3/4 inch exterior grade plywood. All supports shall be fire retardant pressure impregnated wood or steel. Support details shall be submitted to the Engineer for approval.

5.7 The Contractor shall install temporary wooden boxes to protect cables exposed after removing the ducts or conduits. The wood for the boxes and support shall be as called for in Paragraph 5.6. The cables shall be separated as they existed and may be confined within a common enclosure of vertical and horizontal separators; see Sketch "B" attached.

5.8 Only hand labor shall be employed to move or shift cables. The Contractor shall provide safety gloves for the workers handling the high tension and low tension cables, since these cables carry 120 to 27,000 volts and <u>are not</u> de-energized, and are active and alive throughout the construction period.

5.9 Where the Contractor encounters Con Edison transmission lines, refer to Paragraph 4.2.1.b. The initial work would consist of exposing, shifting, protecting and supporting. Shifting of the oil-o-static pipe in the excavated area should be done using six inch (6") wide nylon slings (or equivalent as approved by Con Edison Co.) at each lift point. The pipe or pipes shall be supported in the excavated area as shown on Sketches "A", "C-1" and "C-2" copies attached. Care should be used to prevent or minimize damage to the pipe coating.

6.0 A SUGGESTED METHOD FOR A CONTRACTOR TO BREAK OUT CONDUIT OR DUCT LINES, AND EXPOSE LIVE CABLES.

6.1 Conduit or Duct Lines Not Having a Concrete Encasement.

a. Precast concrete or wooden ducts - break out duct with small hand held tools (<u>powered tools not allowed</u>).

6.2 Concrete Encased Conduit or Duct Lines.

a. From within the manhole, the Contractor determines the approximate location and amount of cables.

b. Using hand-held tools, the Contractor should chip away and remove a short section of the concrete encasement (<u>powered tools not allowed</u>).

c. Using hand-held tools, the Contractor should break away a short section of the conduits or ducts (clay, fiber, plastic, and fiberglass) to expose and locate the cables (<u>power tools not allowed</u>).

d. Should the Contractor elect to use powered tools, the Contractor shall demonstrate the proposed procedure for the Engineer's approval. A suggested method would be to insert metal shields between the cables and the inside duct or conduit bore walls. The only powered tool allowed is a pneumatic powered hand-held chipping gun with a 1-1/8" diameter piston and a 2" stroke, operated at 110 P.S.I. air supply. Using the approved chipping gun, the Contractor starts removing the concrete encasement from the conduit or duct line. After the concrete encasement is removed, the Contractor breaks out the conduits or ducts with hand-held, non-powered tools. The Contractor carefully raises and secures the live cables above the conduit or duct line being broken out.

7.0 PROTECTION AND SUPPORT

7.1 The exposed cables are protected within fire retardant pressure impregnated temporary wooden cable boxes, temporary wooden manholes or temporary wooden service boxes, constructed of either 3/4" exterior grade plywood or 2" tongue and groove lumber.

7.2 A suggested method of supporting the temporary wooden cable boxes is shown on Sketch "A", and construction of the temporary wooden cable boxes is shown on Sketch "B".

7.3 A suggested method for support of the temporary wooden cable manhole is shown on Sketches D-1 and D-2.

7.4 A suggested method of supporting the High Pressure Pipe Cable Line (Oil-O-Static Pipe) is shown on Sketch "A", and the supports are shown on Sketches "C-1", "C-2", in accordance with Con Edison requirements.

7.5 The Contractor shall submit to the Engineer plans and details for the construction of and support of the items outlined in paragraphs 7.1 to 7.4 inclusive.

7.6 The Contractor shall submit to the Engineer for approval, the support of concrete transformer vaults from the decking system, including the transformer therein, with all details.

7.7 All structures temporarily supported from the decking system should be included in the decking system design.

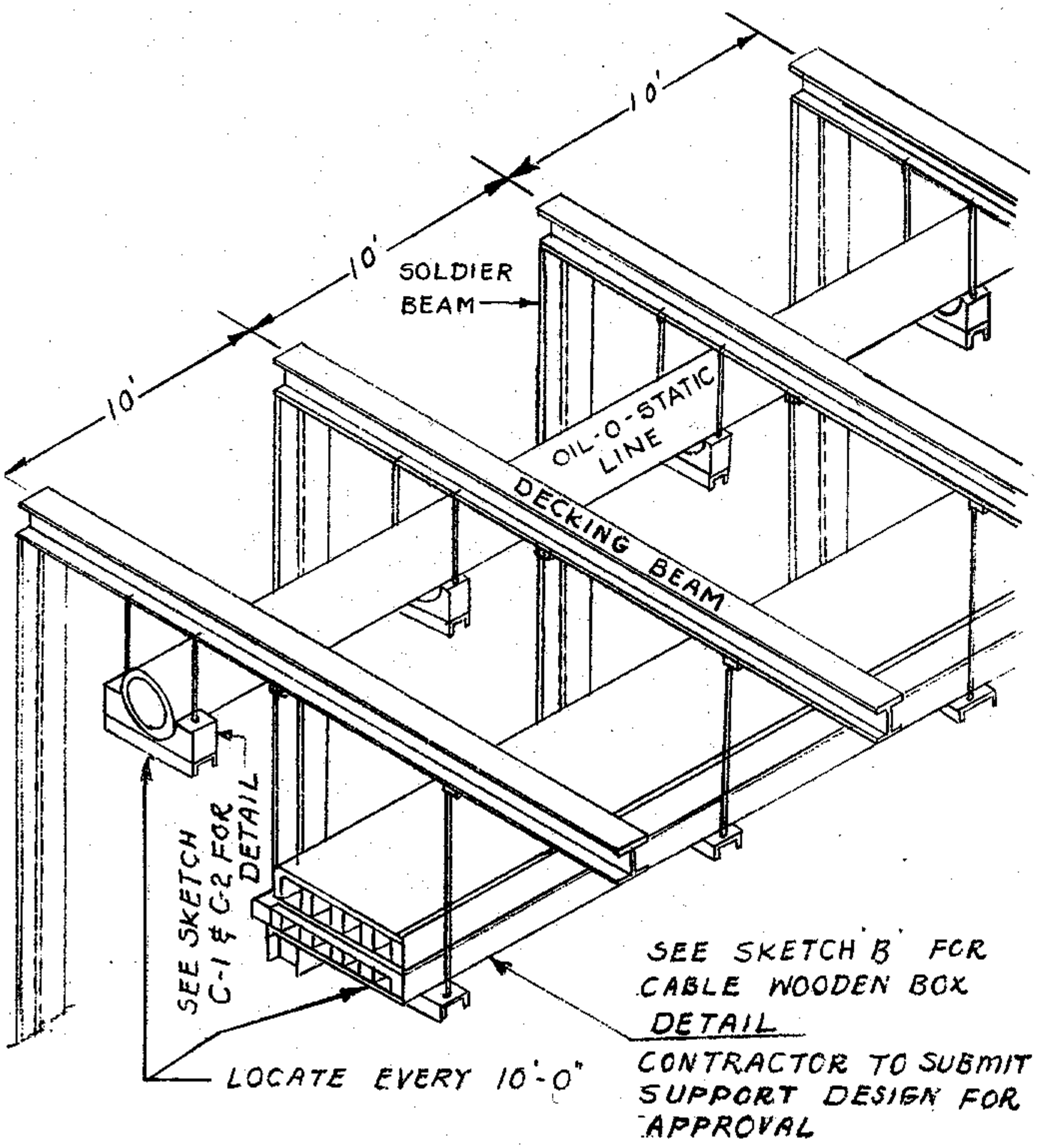
Attachments - Sketches - A, B, C-1, C-2, D-1 and D-2.

0424b

TYPICAL SUPPORT FROM DECKING BEAMS

TEMPORARY SUPPORT

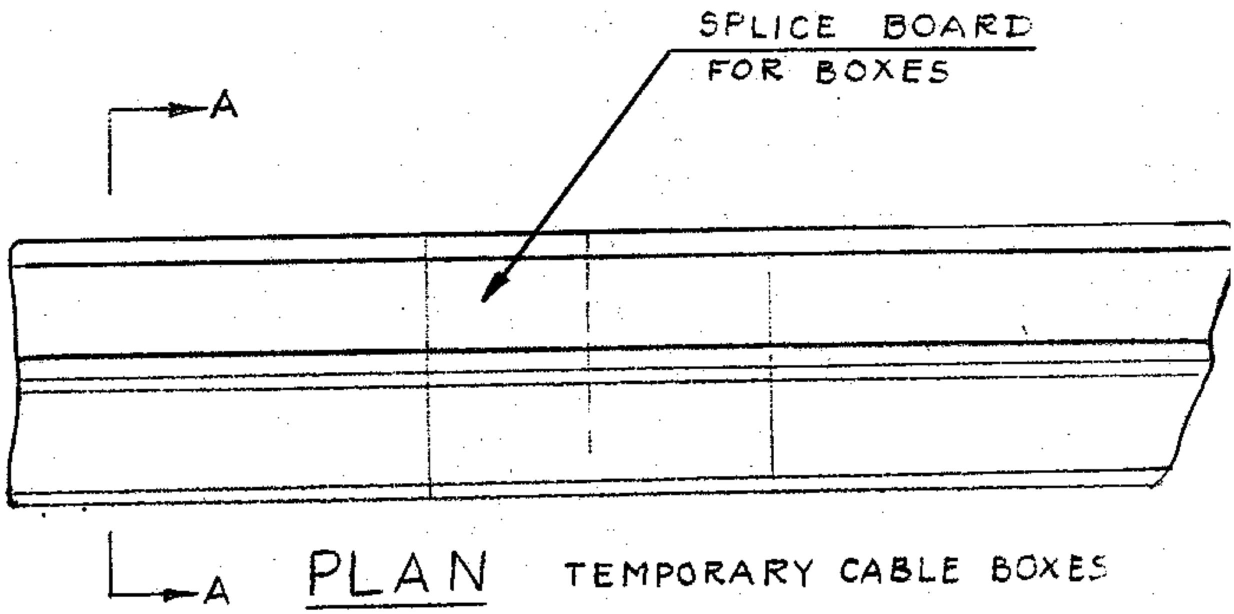
SKETCH “A”

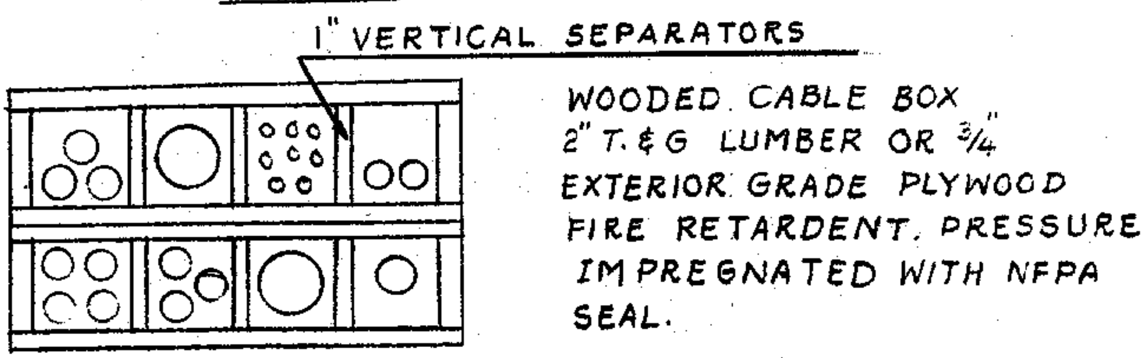


TYPICAL SUPPORT FROM DECKING BEAMS

TEMPORARY WOODEN BOX FOR CABLE PROTECTION

SKETCH "B"



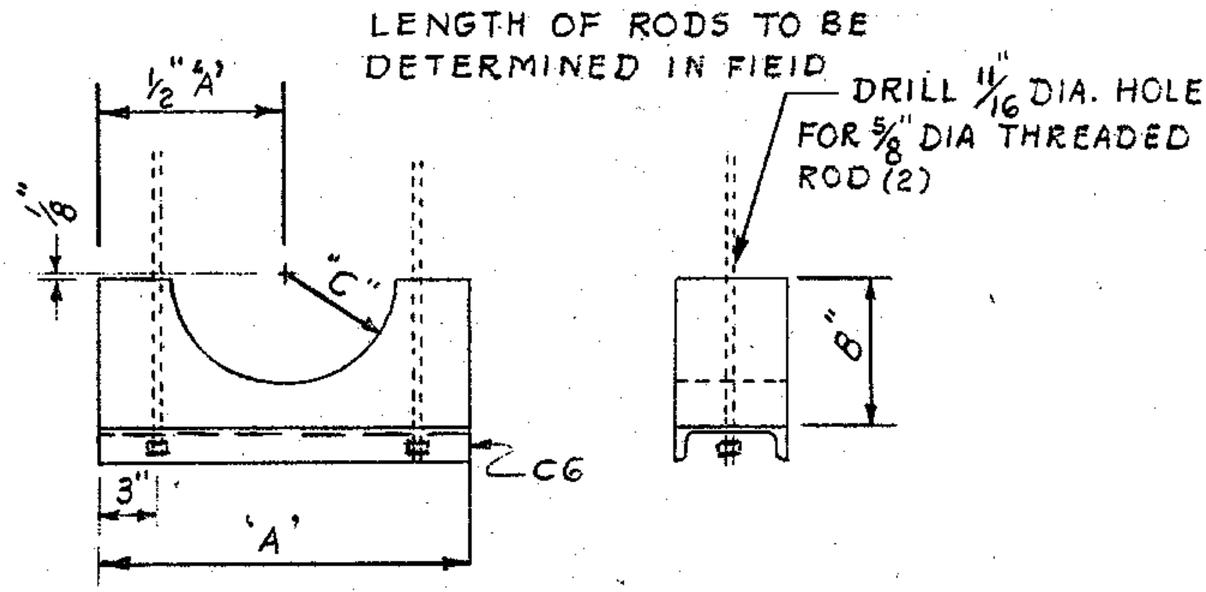


SECTION A-A

TYPICAL SUPPORT FROM DECKING BEAMS

OIL - O - STATIC LINES

SKETCH 'C' (1)



PIPE SIZE	A	B	C
5 9/16"	1'-2"	5¾"	3 1/16"
8 5/8"	1'-6"	7¼"	4 11/16"
10¾"	1'-8"	8¼"	5¾"

(1) SINGLE HP PIPE TYPE CABLE TEMPORARY SUPPORT

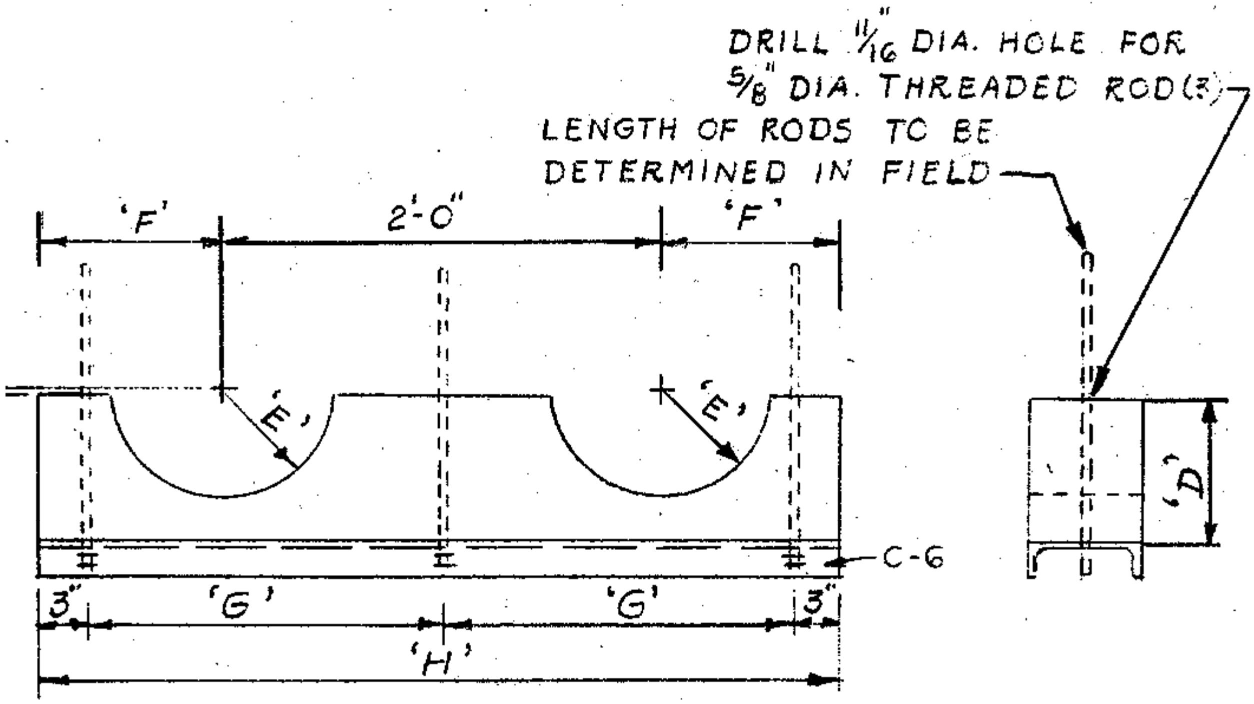
NAT'L STRUCTURAL YELLOW PINE OR FIR

SCALE: 1½ = 1'-0"

TYPICAL SUPPORT FROM DECKING BEAMS

OIL-O-STATIC LINES

SKETCH 'C' ②



PIPE SIZE	D	E	F	G	H
5 9/16"	5 3/4"	3 1/16"	7"	1'-4"	3'-2"
8 5/8"	7 1/4"	4 11/16"	9"	1'-6"	3'-6"
10 3/4"	8 1/4"	5 3/4"	10"	1'-7"	3'-8"

② DOUBLE H.P. PIPE TYPE CABLE TEMPORARY SUPPORT

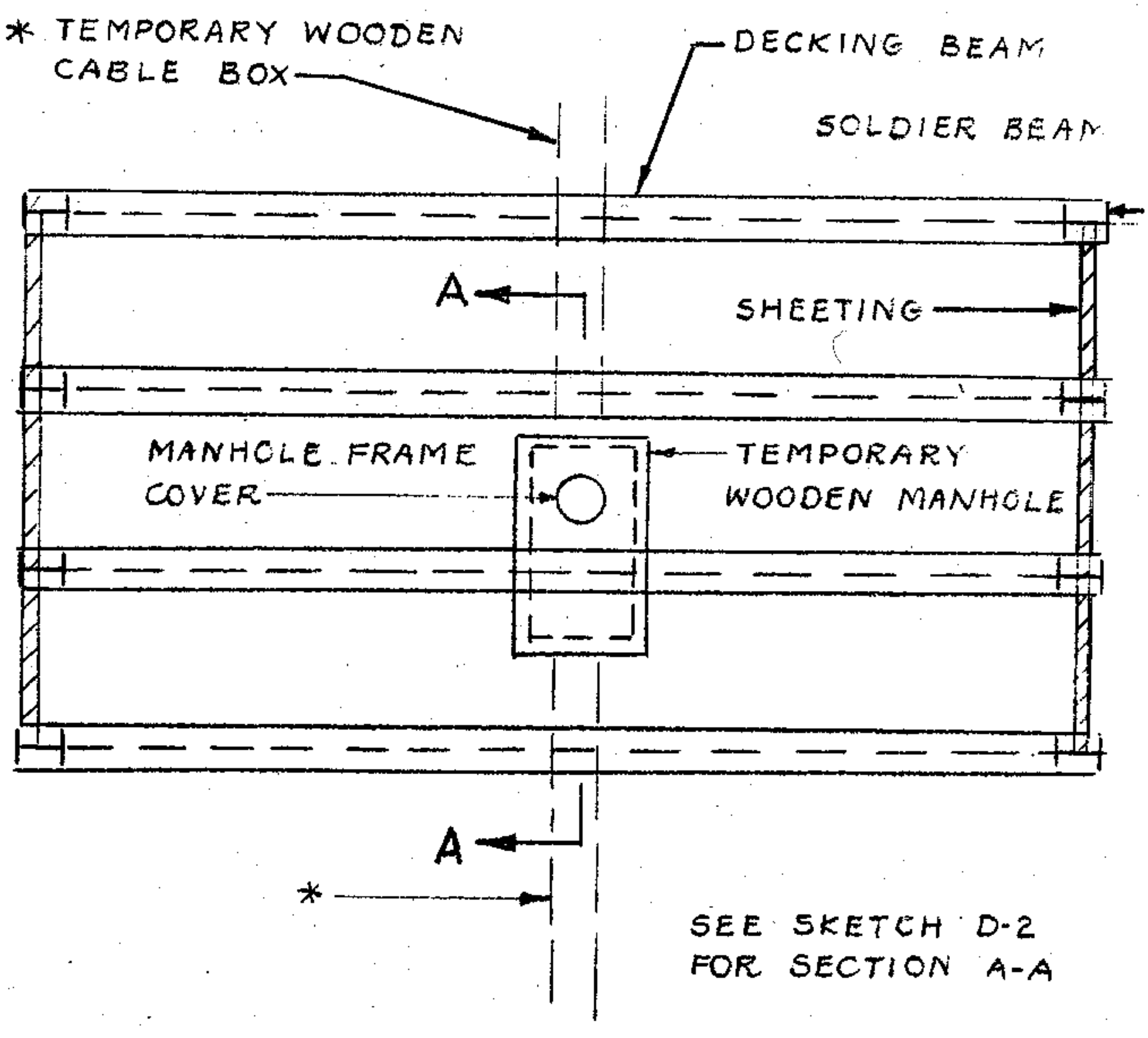
NAT'L STRUCTURAL YELLOW PINE OR FIR

SCALE: 1½" - 1'-0"

TYPICAL

TEMPORARY MANHOLE SUPPORT FROM DECKING BEAMS

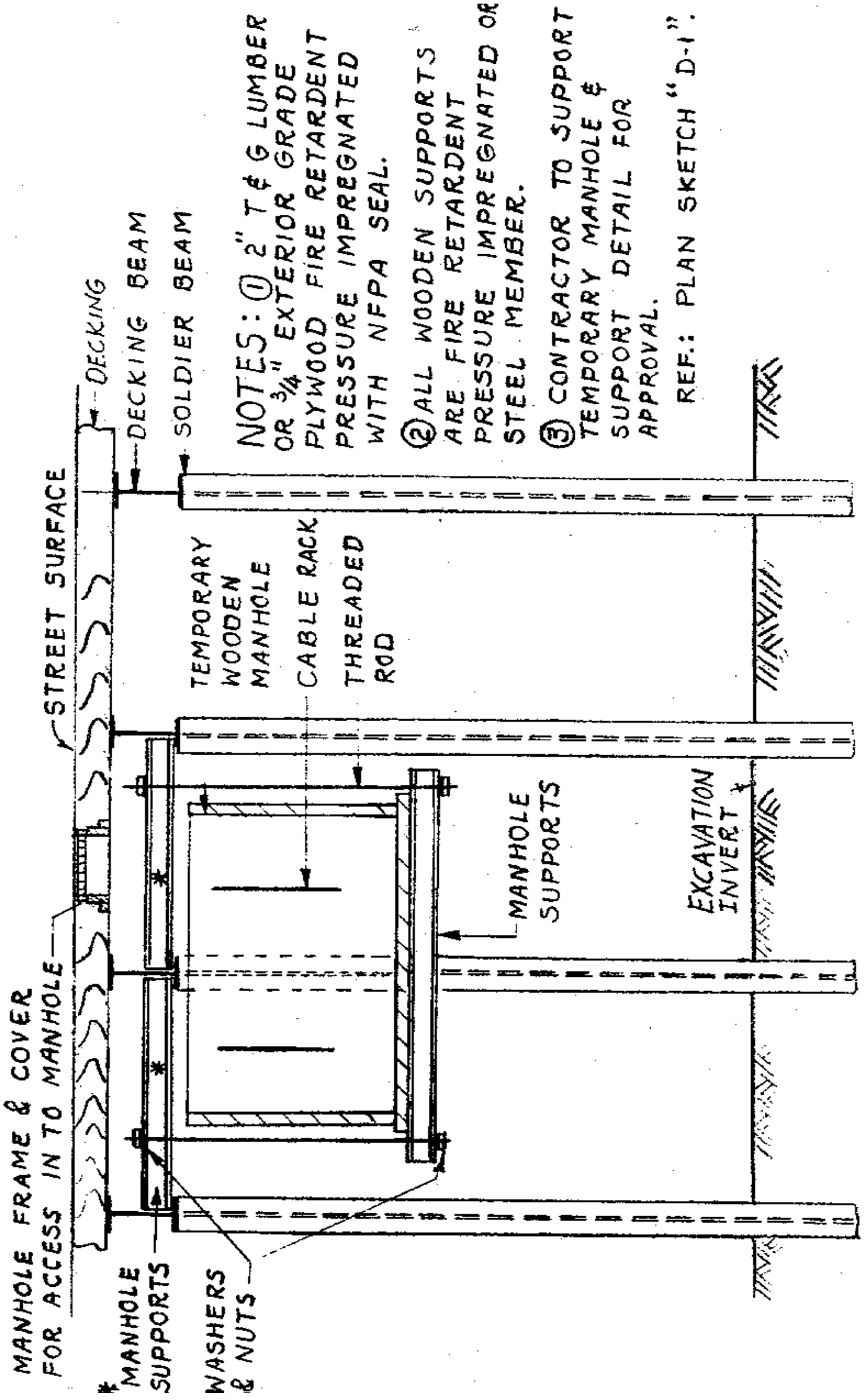
SKETCH "D-1"



PLAN

TEMPORARY MANHOLE SUPPORT

SKETCH 'D-2'



SECTION A-A

SECTION MS

MISCELLANEOUS

MAINTENANCE AND SUPPORT OF THE RAILROAD

This subject combines all methods and measures to protect the existing operating railroad and its appurtenances either temporarily or permanently, in a manner approved by the Engineer, to provide a continuous operation of the railroad.

MAINTENANCES OF THE RAILROAD

Maintenance and protection generally consist of methods of separating and protecting the passengers, the operating railroad and equipment including signals, track, line equipment and power facilities from the elements of weather, debris and other objectionable matters as a result of removing or altering portions of the existing structure.

Horizontal or Vertical Shields are used to provide a tight weatherproof, watertight and fireproof bulkhead or shield properly secured, to protect passengers, employees or other persons from the elements, dust or debris during the construction period.

Vertical shields are required when work is performed close to active tracks if it is not practicable to interrupt the construction operation when trains pass.

When station platform and/or canopy roof are rehabilitated, vertical shields are to be used as a method of "car door blocking" so as not to permit the car doors to open into a work area. This method of car door blocking must be first approved by the Station Department Superintendent and then clearly defined in the specifications.

Horizontal shields are hung below elevated aerial train structures, when structural work is in progress, to protect the vehicular and pedestrian traffic below.

The temporary horizontal shield shall be designed to withstand a minimum live load of one hundred fifteen pounds per square foot (psf), plus a snow load of ten psf. The use of the shields as form work must have the approval of the Engineer and be designed to support the dead load of the poured concrete including the impact for placement.

SUPPORT OF THE RAILROAD

When the existing structures are altered, or members and connections replaced, it may be necessary to temporarily provide timber shores or posting for support. The designs of these members are dictated by the known Transit Authority live and dead loads plus the thrust due to lateral earth pressure in subway, wind on elevated structures and impact loads from the operating railroad. The balance of the structure shall be investigated to check if the loads can safely be dissipated from the new shores.

The existing subway structure must be underpinned before any excavation is performed below the influence line of the existing structure. See the Field Design Manual to determine the extent of underpinning (pp. UP1-UP25). The usual method of pit excavated underpinning of existing walls requires box sheeting and installation of shear in the sides of piers to lock into the adjacent future underpinning piers. The underpinning wall must act as a retaining either exposed or laterally unsupported. This will require a lateral support system of walers plus tiebacks, rakers or struts.

Due to the involved conditions of an operating track when diversion of service is required, a step by step procedure of construction is generally included with the contract documents. In special cases a directed method of construction may be called for in the contract. Temporary support systems which are shown on the drawings to be used when constructing a new structure either adjacent to or below a running railroad shall be generally indicated as "a method of construction."

In order to control and predict settlement or displacement movements, <u>monitoring</u> of structure may be required when construction work effects the operating of adjoining active tracks.

This will include, but not limit, the need of the following actions by the Contractor:

- Inspection of the site before, during and after the construction period.
- Monitoring settlements and displacements, either visually or by instrumentation of the adjacent structures.
- Limits of accepted movements and clear list of stand-by procedures to control and limit the future movements of the existing structure must be included on the Contractor's working drawings.

The need of special instrumentation such as: piezometers, inclinometers, borehole extensometers, tape extensometers, strain gauges, etc., shall be decided by the designer and clearly specified both in the contract drawing and the specifications.

TUNNEL SUPPORT STEEL

Tunnel support steel consisting of ribs and posts are used to temporarily support the rock over the tunnel during the excavation prior to placing the concrete lining. Ribs are usually spaced at 5'-0" c-c and are cold worked to follow the shape of the exterior of the tunnel roof (usually horseshoe, circular or elliptically shaped). The ribs are supported on short posts - which rest on continuous beam grillages on a rock shelf; where the condition of the rock requires, the post may extend down to the tunnel invert. The rock is blocked to the ribs usually at 5'-0" c-c with timber and the end of the rib is blocked to rock with hard wood timber or concrete. If the rock in the roof requires lagging, either timber or steel may be placed between ribs. All ribs and other supports are usually kept beyond the net line of the final concrete lining. Timber is removed prior to placing the lining.

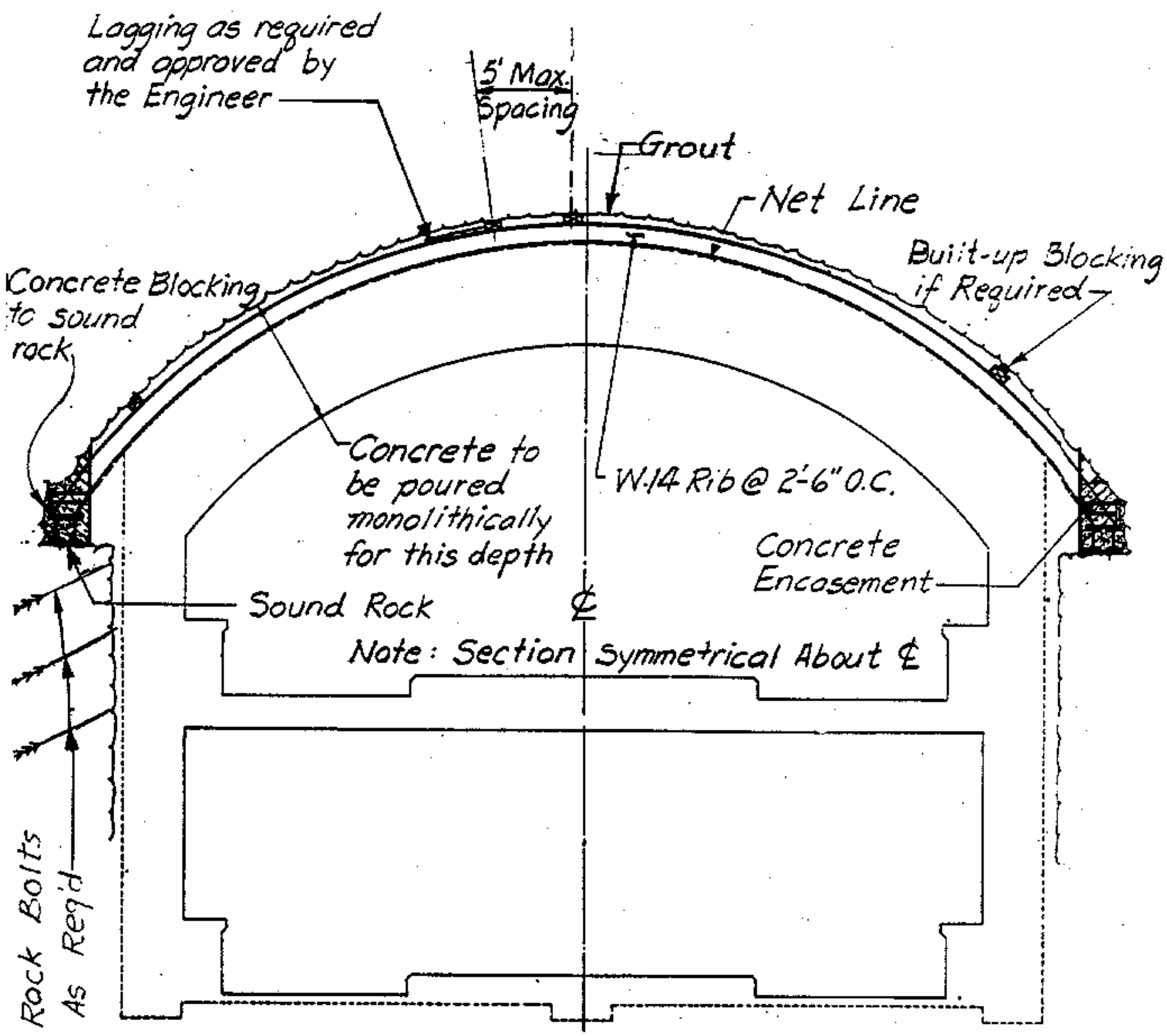
Design loading on the tunnel roof is a function of the RQD (total length of 4" pieces of core over the 5 feet length of boring). Low RQD's will increase design loadings on the roof. Loadings for higher RQD's are a function of the width and shape of the tunnel excavated opening. Loads for bores made with a tunnel boring machine may be reduced by 15% from that shown for drifts made by Drill and Blast. The load for which ribs are designed for may be taken as 75% of that determined from the above procedure. Steel ribs shall be designed for a bending stress of 27 kips. All other members are to be designed using stresses from the A.I.S.C. without any increases.

Structural analysis of the ribs may be made by the method shown in the text "Rock Tunneling with Steel Supports" by Proctor and White, published by the Commercial Shearing and Stamping Co. of Youngstown, Ohio.

The rock walls of the tunnels shall be protected with rock bolts encapsulated in epoxy as required. Such bolts shall be at least one inch diameter and ten feet long. Where required, steel straps shall be placed between bolts. Special consideration shall be given to protecting the shelves on which posts are supported.

The Contractor shall submit his procedure for excavation with heights of excavation lifts, length of rock pull permitted prior to placing ribs and all details of tunnel support steel including all blocking, concreting of footing grillages, grouting of base plates and wall rock bolting.

TUNNEL SUPPORT SYSTEM



TYPICAL STEEL TUNNEL SUPPORT SYSTEM

SIDEWALK VAULTS

Sidewalk vaults are usually extensions of the building cellar beyond the building line, below the sidewalk and typically are used for storage, vehicle parking or portions of mechanical equipment rooms. In one instance, a vault was part of the grand ballroom of a major hotel. Vaults may be multi-story - their number of levels corresponding to the building cellar or basement levels. Sidewalk vaults are built with the permission of the Borough Superintendent.

Sidewalk vaults are constructed by the owner of the building to which they are attached under a permit issued by the Bureau of Highway Operations - DOT. The tax bills sent by the Finance Dept. include the cost of vault rental. Vault rental fees charged by the City are nominal.

When required, a Vault Vacate Notice will be issued by the Bureau of Highway Operations, giving the building owner ten days to remove his property from the vault. The taking of a vault is not a simple process. The issuance and implementation of a vault vacate notice may be a time consuming process. Proof must be furnished that the vault or whatever part may be requested is truly needed, and whether the need is permanent or temporary.

At the time of the "D" Drawing field survey, the Construction Division survey crew will make a detailed survey of the vault if requested to do so by the Designer. Occasionally a more detailed survey of a vault may be made where representatives of CEDD, Construction, and Equipment Division, will inspect and report on the items in the vault which are within their expertise.

The purpose of such an inspection would be to see if the loss of its use to the building owner of the vault is so great as to warrant the Authority redesigning the subway structure which is in interference with all or part of the vault.

When all or a part of a sidewalk vault is required by the Transit Authority, N.Y.C. DOT is informed. They request the Highway Department to issue a Vault Vacate Notice to the building owner to make the vault space available to the Authority.

In general, the Authority will only request vaults when required to permit the installation of T.A. facilities. If possible, only a portion of a vault will be taken or the vault may be taken on a temporary basis.

During construction of a T.A. structure, vaults may be used temporarily, after which they will be returned to the building owner after the T.A. construction is completed. The vault owner is responsible to maintain his building operation. Closure walls will be installed at the limits of the vault to protect it from the elements.