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Ladislav Lamboj

H. Y. Fang

Earth Pressures and Retaining Structures

Comparison of Maximum Moment, Tie Rod Force and Embedment Depth of Anchored Sheet Pile

by

Ladislav Lamboj
H. Y. Fang

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Fritz Engineering Laboratory
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Lehigh University
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ABSTRACT

Comparisons of theoretical embedment depth, tie rod force and positive maximum bending moment by various existing methods such as free-earth, Rowe, Blum, Anderson, Tschebotarioff, Turabi and Balla methods are carried out for 5, 10, 15 and 20 ft. high bulkheads with the tie rod placed at the ground surface under three different soil conditions such as loose sand, medium sand and dense sand.

It was concluded that the higher the angle of internal friction, the closer are results of all presented methods. In the case of flexible wall driven to the medium sand there is no significant difference between Blum, Anderson and Tschebotarioff methods, but for loose sand it is very important to establish the rigidity of the wall.

Due to large scatter, no positive conclusions were drawn in an attempt to compare these theoretical results with field measurements conducted by Lehigh University.

1. INTRODUCTION

The purpose of this report is to conduct theoretical comparison between customary design methods on anchored sheet pile walls and field measurements carried out by Lehigh University.

In previous literature Rimstad (1940) carried out theoretical comparison for a bulkhead of height H = 23 ft. The sheet pile wall was driven through three different layers of sand. Ground water level was considered 8.2 ft. under the ground surface and water level in excavation was 13.2 ft. above the dredge line. The anchor rod was placed 4.9 ft. under ground surface. There was surcharge loading. Rimstad compared fixed-earth method, equivalent beam method with point of inflection located 0.1 H below the dredge line, free-earth method, Danish regulations and Ohde's recommendation under the conditions that there is a safety factor for the embedment depth equal to 2 and no anchor yielding.

In the case of the embedment depth, the equivalent beam method gives the largest value, followed by fixed-earth method, free-earth method, Ohde's recommendation and Danish regulations. The embedment depth determined with the equivalent beam method is roughly 330% of the value obtained with Danish regulations.

In the case of the tie rod force, Ohde's recommendation gives the largest value, followed by free-earth method, fixed-earth method, equivalent beam method and Danish regulations. The tie rod force computed with Ohde's recommendation is roughly 160% of the value obtained with Danish regulations.

In the case of the positive maximum bending moment, the free-earth method gives the largest value, followed by fixed-earth method, equivalent beam method, Ohde's recommendation and Danish regulations. The positive maximum bending moment computed with free-earth method is roughly 290% of the value obtained with Danish regulations.

T. Edelman et al (1958) published comparative sheet piling calculations. Five different types of wall, each with a surcharge of 0,4 and 12 Mp/m^2 were treated according to the methods of Tschebotarioff, Schütte, Rowe, Blum and the Danish Rules. The soil was supposed to consist of homogeneous sand with an angle of internal friction $\Phi = 30°$ and a unit weight 1.7 Mp/m^3 (saturated 2.0 Mp/m^3).

In this report a theoretical comparison is carried out between free-earth method, Rowe method, equivalent beam method proposed by Blum, equivalent beam method proposed by Anderson, equivalent beam method proposed by Tschebotarioff and Turabi and Balla method. The procedure which has to be followed to use these methods is not explained in detail in this report and the reader is directed to original literature.

Walls of 5, 10, 15 and 20 ft. high with the tie rod located at the ground surface were chosen for theoretical study since field measurement was performed on the same geometrical type walls. Behavior of these walls is treated under three different conditions representing different relative densities of sand. Physical properties of sands were considered according to Peck et al (1953), Terzaghi (1955) and Meyerhof (1956). In the first case, the walls are considered to be driven into unlimited layer of loose sand ($\Phi = 32°$, $\gamma_m = 110$ lb/cu. ft., m = kip/cu. ft.); secondly, the walls are considered to be driven into unlimited layer of medium sand ($\Phi = 35°$, $\gamma_m = 110$ lb/cu. ft., m = 16 kip/cu. ft.) and finally the walls are considered to be driven into unlimited layer of dense sand ($\Phi = 38°$, $\gamma_m = 110$/cu. ft., m = 40 kip/cu. ft.). There was no consideration of ground water level and no yielding of anchor is taken into account.

In the case of the Rowe or Turabi and Balla methods, DP-2 sheet pile is considered. DP-2 sheet pile has properties as follows: nominal width 16 in., moment of inertia $I = 53$ in.^{4}, elastic modulus $E = 29.6 \times 10^3$ kip in.$^{-2}$.

In this report there is no consideration of anchor design and no consideration of stability of the entire system.

2. DESCRIPTION OF DESIGN METHODS

Design methods on sheet pile walls can be divided into four main groups as follows:

1) Free-Earth Method
2) Fixed-Earth Method
3) Limit Design Method proposed by Brinch-Hansen (1953)
4) Empirical Methods

Other methods are modifications of previous ones.

This report will consider free-earth method, Rowe method, Blum method, Anderson method, Tschebotarioff method (Fig. 1), and Turabi and Balla method. The substantial difference between free-earth and fixed-earth methods is in the assumption of the pile rigidity. The free-earth method considers a sheet pile as a rigid body, the fixed-earth method considers a sheet pile as a flexible body, however, Rowe, Turabi and Balla methods only quantitatively distinguish sheet pile flexibility.

The free-earth method is usually recommended in loose silty-sand deposits. Rowe method and all modifications of the fixed-earth method are usually used for uniform medium-dense to dense silty sand or sand deposits.

2.1 Free-Earth Method

1) The theoretical embedment depth is determined from an assumption that a moment at the tie rod level must be equal to zero. The general equation for the embedment depth is as follows:

$$2(K_a-K_p)\left(\frac{D}{H}\right)^3 + 3(2K_a-K_p)\left(\frac{D}{H}\right)^2 + 6K_a\left(\frac{D}{H}\right) + 2K_a = 0$$

The recommended embedment depth is $D' = 1.4D$.

2) The value of the tie rod force is determined from an assumption that the equilibrium in the horizontal direction must be satisfied. The general equation for solving the tie rod force is

$$T = \frac{1}{2}\gamma_m\left[(H+D)^2 K_a - D^2 K_p\right]$$

3) The value of the positive maximum bending moment is at the point which has the distance x from the top of sheet pile given by the equation

$$x = \sqrt{\frac{2T}{\gamma_m K_a}}$$

The positive maximum bending moment is computed from the equation

$$M_{max} = Tx - 0.167\ \gamma_m\ x^3\ K_a$$

Obtained results are presented in Fig. 2, 3, 4, 5 and in Table 1.

2.2 Rowe Method (1952)

The procedure for obtaining all design elements is the same as in the free-earth method, however, the reduction of the positive maximum bending moment is provided using a flexibility number ρ

$$\rho = \frac{(H+D')^4}{EI},$$

a coefficient α which expresses the length of the pile above the dredge line as a proportion of the entire length of the pile, a coefficient β which expresses the distance below ground surface, where the tie rod is located, as a proportion of the entire length of the pile and soil type.

Obtained results are presented in Fig. 18 and in Table 5.

2.3 Blum Method (1931)

1) The location of the point of the inflection I,

a distance b below the dredge line, is carried out after Verdeyen and Roisin (1961) as a function of H and ϕ and was established using $K_p = 2/K_a$.

2) The reaction B of the equivalent beam is solved from the equation

$$B = \frac{E_{a1}\, r_{a1} + E_{a2}\, r_{a2} + E_{a3}\, r_{a3}}{H + b},$$

where

$$E_{a1} = \frac{1}{2} \gamma_m H^2 K_a \qquad r_{a1} = \frac{2}{3} H$$

$$E_{a2} = \gamma_m HK_a \frac{a - b}{a} b \qquad r_{a2} = H + \frac{b}{2}$$

$$E_{a3} = \frac{1}{2} \gamma_m \frac{b^2}{a} HK_a \qquad r_{a3} = H + \frac{b}{3}$$

$$a = \frac{H\, K_a}{K_p - K_a}$$

3) The tie rod force is solved from the equation

$$T = E_{a1} + E_{a2} + E_{a3} - B$$

4) The value of the positive maximum bending moment is at the point which has the distance x from the top of the

sheet pile given by the equation

$$x = \sqrt{\frac{2T}{\gamma_m K_a}}$$

The maximum bending moment is computed from the equation

$$M_{max} = Tx - 0.167\ \gamma_m\ x^3\ K_a$$

5) The embedment depth is solved from the equations

$$D = Y + b$$

$$D' = 1.2D$$

Obtained results are presented in Fig. 6, 7, 8 and in Table 2.

2.4 Anderson Method (1956)

1) The location of the point of the inflection is at the point where the lateral earth pressure is equal to zero.

This point is located at the distance a below the dredge line

$$a = \frac{H\,K_a}{K_p - K_a}$$

2) The reaction B of the equivalent beam is solved from the equation

$$B = \frac{E_{a1}\,r_{a1} + E_{a2}\,r_{a2}}{H + a},$$

where

$$E_{a1} = \frac{1}{2}\gamma_m H^2 K_a \qquad r_{a1} = \frac{2}{3}H$$

$$E_{a2} = \frac{1}{2}\gamma_m aH K_a \qquad r_{a2} = H + \frac{1}{3}a$$

3) The tie rod force is solved from the equation

$$T = E_{a1} + E_{a2} - B$$

4) The value of the maximum bending moment is at the point which has the distance x from the top of sheet pile given by the equation

$$x = \sqrt{\frac{2T}{\gamma_m K_a}}$$

The maximum positive bending moment is computed from the equation

$$M_{max} = Tx - 0.167\ \gamma_m\ x^3\ K_a$$

5) The embedment depth is solved from the equations

$$D = Y + a$$

$$D' = 1.2D$$

where

$$Y = \sqrt{\frac{6B}{\gamma_m\ (K_p - K_a)}}$$

Obtained results are presented in Fig. 9, 10, 11 and in Table 3.

2.5 Tschebotarioff Method (1951)

1) The point of the inflection is located at the dredge line.

2) The coefficient of the active earth pressure

$$K_a' = 0.33 \times 0.9 = 0.297$$

3) The reaction B of the equivalent beam is solved from the equation

$$B = \frac{1}{3} \gamma_m H^2 K_a'$$

4) The tie rod force is solved from the equation

$$T = \frac{1}{2} \gamma_m H^2 K_a - B$$

5) The value of the positive maximum bending moment is at the point which has the distance x from the top of sheet pile given by the equation

$$x = \sqrt{\frac{2T}{\gamma_m K_a}}$$

The maximum bending moment is computed from the equation

$$M_{max} = Tx - 0.167 \gamma_m x^3 K_a$$

6) The embedment depth is solved from the equation

$$D' = 0.43 H$$

Obtained results are presented in Fig. 12, 13, 14 and in Table 4.

2.6 Turabi and Balla Method (1968)

1) The embedment depth is determined using the principle of free-earth method.

2) The tie rod force and the positive maximum bending moment are determined taking into account the depth of anchorage expressed as a ratio β of the depth of anchorage to the total pile length and the flexural parameter κ

$$\kappa = \frac{EI}{Kh^3} = \frac{EI}{0.1mh^4} ,$$

where

$$h = {}^{D'}/_{5} ,$$

$$K = 0.1\ hm$$

The tie rod force is computed from the equation

$$T = \Omega_{\gamma}\ P_E ,$$

where

$$P_E = \frac{1}{2}\gamma_m H^2 K_a$$

The positive maximum bending moment is computed from the equation

$$M_{red} = 2M_E \sqrt{\left(\frac{T}{P_E}\right)^3} ,$$

where

$$M_E = \frac{1}{6}\gamma_m H^3 K_a$$

Obtained results are presented in Table 6.

3. FIELD MEASUREMENTS

Field measurements were carried out on the sheet-pile wall at the site in Martins Creek, Pennsylvania by Lehigh University (Fang, Brewer, 1968; Brewer, Fang, 1969). More detailed information is presented in these publications. The length of the sheet-pile wall was 30 feet and the total length of the arch piles DP-2 was 30 feet and they were driven to a depth of 25 feet. The tie rods were located at the ground surface. Four piles were instrumented by foil-type rosettes (Fig. 20). Rosettes were protected by epoxy covering with the additional protection of a steel shoe that was welded to the sheet pile.

The excavation in front of the wall was performed in four stages (Fig. 19). Initially, a 5 ft. excavation was made and all gages were read. The excavation was left for one week at the end of that time all gages were read again. The next 5 ft. stage of excavation was then made and gage readings were taken one week after excavation. This sequence of events was repeated until the excavation reached the 20 ft. level. Strain readings are presented in graphical form in Fig. 21 to Fig. 24. These readings have served for the evaluation of the moment distribution along the pile. The tie rods were calibrated in Fritz Engineering Laboratory.

The major mechanical-physical properties of sand on the site were as follows: in-place unit weight γ_m = 117 lb/cu. ft., angle of internal friction $\Phi = 38°$.

4. DISCUSSION

4.1 Theoretical Results

4.1.1 Embedment Depth

Figure 15 clearly shows that the embedment depth designed by free-earth (Rowe and Turabi and Balla methods use this same value) and Tschebotarioff methods are shorter than the embedment depth designed by Blum or Anderson methods. The difference becomes smaller with the increasing of the angle of internal friction, ranging approximately from 100-170% for the angle of internal friction $\Phi = 32°$, 100-149% for $\Phi = 35°$ and 100-153% for $\Phi = 38°$.

4.1.2 Tie Rod Force

Theoretical results for assumed bulkheads are shown in Fig. 16. The free-earth method gives for all magnitudes of the angle of internal friction the highest values of the tie rod force. For $\Phi = 35°$ there is no significant difference whether using Blum, Anderson or Tschebotarioff methods. Considering a 20 ft. high bulkhead, the tie rod force varies from 100-154% for $\Phi = 32°$, when $\Phi = 35°$ the range of tie rod force is from 100 to 135% and in the case of $\Phi = 38$% there is variation from 100 to 135%.

4.1.4 Maximum Bending Moment

The comparison of the positive maximum bending moment is carried out in a graphical form in Fig. 17 and 18. Due to evaluation of the tie rod force it can be observed that the same trend of differences also occurs for positive maximum bending moment.

In order to make a comparison for all presented methods including Rowe's, Turabi's and Balla's and in agreement with field testing, DP-2 steel sheet pile was chosen. In the case of a 20 ft. high bulkhead driven into cohesionless soil with $\Phi = 32°$ there is a range of value of the positive maximum bending moment from 100 to 188%, for $\Phi = 35°$ positive maximum bending moment varies from 100 to 194% and in the case of $\Phi = 38°$ the range is from 100 to 164%.

The author's results are in very good agreement with theoretical results obtained by Rimstad. It is believed that there are numerical mistakes in Edelman et al work, especially in the case of embedment depth, however, because there is no mathematical formulation of problems, the author could not make a proof of computation.

4.2 Field Measurements

Field measurements conducted by Lehigh University in 1968 were generally described in Chapter 3. The main goal of field measurements was to obtain values of the tie rod force and values of shear strength transferring across interlocks. Carefully studying Fig. 15 for $\Phi = 38°$, one can make a conclusion that only comparison of theoretical results and field measurements for 15 and 20 ft. high bulkhead can be made when the embedment depth in the field is very close to theoretical results.

The bulkhead was not treated from the standpoint of view of stability which means that there was no treatment of the influence of the embedment depth on collapse of whole system. For this reason, comparison cannot be determined between theoretical results and field measurements in the case of embedment depth.

Figure 16 shows theoretical results and field measurements for the tie rod force. The essential difference between measured values and theoretical values can be explained by the breaking of the tie rod when excavation in front of the wall was 5 ft. deep. At this time, a considerable horizontal movement probably occured which brought a decreasing of active earth pressure and thus decreasing the tie rod force. It is not possible to carry out an entire comparison because there are no records about temperature during calibration of the tie rod and even records about temperature during testing in situ are not complete.

Bending moment computations are based on strain records. Sheet piles are considered as single acting units. Figures 25 and 26 show moment diagrams due to previously described theories and points which were evaluated from field data using elastic theory. There is considerable scatter. Numerical values of strain were obtained from reading of strain gages which were located near interlocks. These values could be considerably influenced by presence of soil particles in interlocks which can explain the considerable scatter beyond theoretical values. The next very important factor is relative density of sand. In certain areas with extremely high relative density which can be caused by driving the sheet pile can act with surrounding soils as a unit and thus considerable shifting of neutral axis can occur.

5. CONCLUSION

From the results of this investigation, the following conclusions may be drawn:

1) In theoretical search the higher the angle of internal friction, the closer are results of all presented methods. In the case of flexible wall driven to medium sand there are no significant differences between Blum, Anderson and Tschebotarioff methods. For loose sand it is very important to establish whether bulkhead is flexible or rigid.

2) Due to scatter in moment values evaluated from field measurements it is believed that composite action between soil and the piling occurs, however, further investigation of the soil-structure interaction is necessary in order to more clearly understand this phenomenon.

3) It is believed that some investigation should be made for establishing the influence of soil particles in the interlocks on local stresses and strains.

4) In this report any recommendation can be given for using some design method because of considerable scatter in evaluation of field data.

6. ACKNOWLEDGEMENTS

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7. NOMENCLATURE

B	reaction of equivalent beam
D	theoretical embedment depth
D'	recommended embedment depth
D_r	relative density
E	modulus of elasticity
E_{ai}	resultant of active earth pressure per unit length
H	wall height above dredge line
I	moment of inertia, point of inflection
K	spring stiffness coefficient for the first spring located 0.5h under the dredge line
K_a	coefficient of active earth pressure after Coulomb
K'_a	coefficient of active earth pressure after Tschebotarioff
K_p	coefficient of passive earth pressure after Coulomb
M_E	moment of earth pressure for DP-2 about the dredge level
M_{max}	positive maximum bending moment on sheet-pile wall per unit length
M_o	positive maximum bending moment for DP-2 after Free-Earth Method
M_{red}	positive maximum bending moment for DP-2 after Rowe, Turabi and Balla methods

N	standard penetration number
P_E	resultant of active earth pressure for DP-2 above dredge line
T	tie rod force
Y	length of the second simply supported beam
a	distance below the dredge line where lateral earth pressure is equal to zero
b	distance below the dredge line locating a point of inflection
h	distance between spring supports
m	coefficient of horizontal subgrade reaction at the toe of the sheet pile
r_{ai}	arm of force E_{ai}
x	distance below the top of the sheet pile where positive maximum bending moment is
Φ	angle of internal friction
Ω_γ	dimensionless quantity expressing the effect of the earth pressure P_E on the tie rod force
α	ratio of the height of the supported earth mass and of the total sheet-pile length
β	ratio of the depth of anchorage to the total pile length
γ_m	unit weight of soil
κ	flexural parameter
ρ	flexibility number

8. TABLES AND FIGURES

γ_m	ϕ, K_a, K_p	H	D	D'=1.4D	T	x	M_{max}
pcf	o	ft.	ft.	ft.	kip/ft.	ft.	kip-ft/ft.
110.0	32	5	1.78	2.49	0.21	3.52	0.49
	0.3072	10	3.56	4.99	0.84	7.04	3.92
	3.2544	15	5.34	7.48	1.88	10.56	13.24
		20	7.13	9.97	3.35	14.08	31.39
	35	5	1.50	2.09	0.17	3.42	0.40
	0.2710	10	2.99	4.19	0.70	6.85	3.19
	3.6902	15	4.49	6.28	1.57	10.27	10.76
		20	5.98	8.38	2.80	13.70	25.51
	38	5	1.26	1.76	0.15	3.34	0.32
	0.2378	10	2.52	3.52	0.58	6.69	2.60
	4.2025	15	3.78	5.28	1.32	10.03	8.79
		20	5.03	7.05	2.34	13.38	20.84

TABLE 1 - FREE-EARTH METHOD: EMBEDMENT DEPTH D or D', TIE ROD FORCE T, MAXIMUM BENDING MOMENT M_{max}

γ_m	ϕ, K_a, K_p	H	a	b	B	T	Y	D	D'=1.2D	x	M_{max}
pcf	o	ft.	ft.	ft.	kip/ft.	kip/ft.	ft.	ft.	ft.	ft.	kip-ft/ft.
110.0	32	5	0.52	0.25	0.30	0.15	2.80	3.05	3.66	3.01	0.31
	0.3072	10	1.04	0.50	1.20	0.61	5.60	6.10	7.32	6.03	2.47
	3.2544	15	1.56	0.75	2.71	1.38	8.40	9.15	10.98	9.05	8.34
		20	2.08	1.00	4.81	2.46	11.20	12.20	14.64	12.06	19.76
	35	5	0.40	0.12	0.26	0.13	2.48	2.60	3.13	2.94	0.25
	0.2710	10	0.79	0.25	1.04	0.52	4.96	5.21	6.25	5.89	2.03
	3.6902	15	1.19	0.38	2.33	1.16	7.44	7.81	9.37	8.83	6.84
		20	1.58	0.50	4.14	2.07	9.92	10.42	12.50	11.78	16.22
	38	5	0.30	0	0.22	0.11	2.24	2.24	2.69	2.87	0.21
	0.2378	10	0.60	0	0.88	0.43	4.49	4.49	5.38	5.74	1.65
	4.2025	15	0.90	0	1.97	0.97	6.73	6.73	8.08	8.61	5.57
		20	1.20	0	3.50	1.73	8.97	8.97	10.77	11.49	13.21

TABLE 2 - BLUM METHOD: EMBEDMENT DEPTH D', TIE ROD FORCE T, MAXIMUM BENDING MOMENT M_{max}

γ_m	ϕ, K_a, K_p	H	a	B	T	Y	D	D'=1.2D	x	M_{max}
pcf	o	ft.	ft.	kip/ft.	kip/ft.	ft.	ft.	ft.	ft.	kip-ft/ft.
110.0	32	5	0.52	0.30	0.17	2.34	2.86	3.44	3.17	0.36
	0.3072	10	1.04	1.18	0.68	4.68	5.72	6.87	6.35	2.88
	3.2544	15	1.56	2.67	1.53	7.02	8.59	10.31	9.52	9.71
		20	2.08	4.74	2.72	9.37	11.45	13.74	12.69	23.02
	35	5	0.40	0.26	0.14	2.03	2.43	2.91	3.11	0.30
	0.2710	10	0.79	1.03	0.58	4.06	4.85	5.82	6.21	2.38
	3.6902	15	1.19	2.32	1.30	6.09	7.28	8.73	9.32	8.04
		20	1.58	4.13	2.30	8.12	9.70	11.64	12.43	19.06
	38	5	0.30	0.22	0.12	1.76	2.06	2.47	3.06	0.25
	0.2378	10	0.60	0.90	0.49	3.52	4.11	4.94	6.11	1.99
	4.2025	15	0.90	2.02	1.10	5.27	6.17	7.41	9.16	6.70
		20	1.20	3.59	1.95	7.03	8.23	9.88	12.22	15.90

TABLE 3 - ANDERSON METHOD: EMBEDMENT DEPTH D', TIE ROD FORCE T, MAXIMUM BENDING MOMENT M_{max}

γ_m	K'_a	H	D'	T	x	M_{max}
pcf		ft.	ft.	kip/ft.	ft.	kip-ft/ft.
110.0	0.297	5	2.15	0.14	2.93	0.27
		10	4.30	0.54	5.75	2.07
		15	6.45	1.22	8.64	7.02
		20	8.60	2.18	11.53	16.77

TABLE 4 - TSCHEBOTARIOFF METHOD: EMBEDMENT DEPTH D', TIE ROD FORCE T, MAXIMUM BENDING MOMENT M_{max}

ϕ	H	H+D'	α	$\rho=(H+D')^4/EI$	log ρ	$\frac{M_{red}}{M_o}$	M_o	M_{red}
o	ft.	ft.		ft^2/kip			kip-ft.	kip-ft.
32°	5	7.5	0.67	2.0169×10^{-6}	-5.695 294	1.00	0.604	0.604
	10	15.0	0.67	3.2270×10^{-5}	-4.491 201	1.00	4.985	4.985
	15	22.5	0.67	1.6048×10^{-4}	-3.794 525	0.65	18.529	12.050
	20	30.0	0.67	5.0947×10^{-4}	-3.292 856	0.50	47.455	23.727
35°	5	7.1	0.70	1.6198×10^{-6}	-5.790 485	1.00	0.592	0.592
	10	14.2	0.70	2.5917×10^{-5}	-4.586 365	1.00	4.226	4.226
	15	21.3	0.70	1.3120×10^{-4}	-3.882 066	0.72	13.836	9.958
	20	28.4	0.70	4.1467×10^{-4}	-3.382 266	0.52	33.912	17.636
38°	5	6.8	0.74	1.3629×10^{-6}	-5.865 504	1.00	0.365	0.365
	10	13.5	0.74	2.1172×10^{-5}	-4.674 279	1.00	3.612	3.612
	15	20.3	0.74	1.0825×10^{-4}	-3.965 773	0.81	11.397	9.224
	20	27.0	0.74	3.3876×10^{-4}	-3.470 057	0.59	28.993	17.102

TABLE 5 - ROWE METHOD, REDUCED MOMENT FOR DP-2 M_{red}

Φ	H	D'	H+D'	h	κ	log κ	α	Ω_γ	P_E	T	M_E	M_{red}
o	ft.	ft.	ft.	ft.					kip	kip	kip-ft.	kip-ft.
32°	5	2.5	7.5	0.5	3.488×10^5	5.542	0.67	0.4897	0.5632	0.2758	0.939	0.644
	10	5.0	15.0	1.0	2.179×10^4	4.338	0.67	0.4897	2.2527	1.1031	7.509	5.146
	15	7.5	22.5	1.5	4.304×10^3	3.634	0.67	0.4747	5.0687	2.4061	25.343	16.577
	20	10.0	30.0	2.0	1.362×10^3	3.134	0.67	0.4592	9.0110	4.1378	60.073	37.386
35°	5	2.1	7.1	0.4	2.660×10^5	5.425	0.70	0.4747	0.4968	0.2358	0.828	0.542
	10	4.2	14.2	0.8	1.662×10^4	4.221	0.70	0.4747	1.9873	0.9434	6.624	4.333
	15	6.3	21.3	1.3	2.384×10^3	3.377	0.70	0.4509	4.4714	2.0162	22.355	13.537
	20	8.4	28.4	1.7	8.152×10^2	2.911	0.70	0.4296	7.9491	3.4149	52.991	29.842
38°	5	1.8	6.8	0.4	1.064×10^5	5.027	0.74	0.4551	0.4359	0.1984	0.727	0.446
	10	3.5	13.5	0.7	1.134×10^4	4.055	0.74	0.4509	1.7438	0.7863	5.813	3.520
	15	5.3	20.3	1.1	1.860×10^3	3.270	0.74	0.4260	3.9236	1.6714	19.618	10.909
	20	7.0	27.0	1.4	7.090×10^2	2.851	0.74	0.4018	6.9753	2.8027	46.502	23.687

TABLE 6 - TURABI AND BALLA METHOD: TIE ROD FORCE T, MAXIMUM BENDING MOMENT M_{red} FOR DP-2

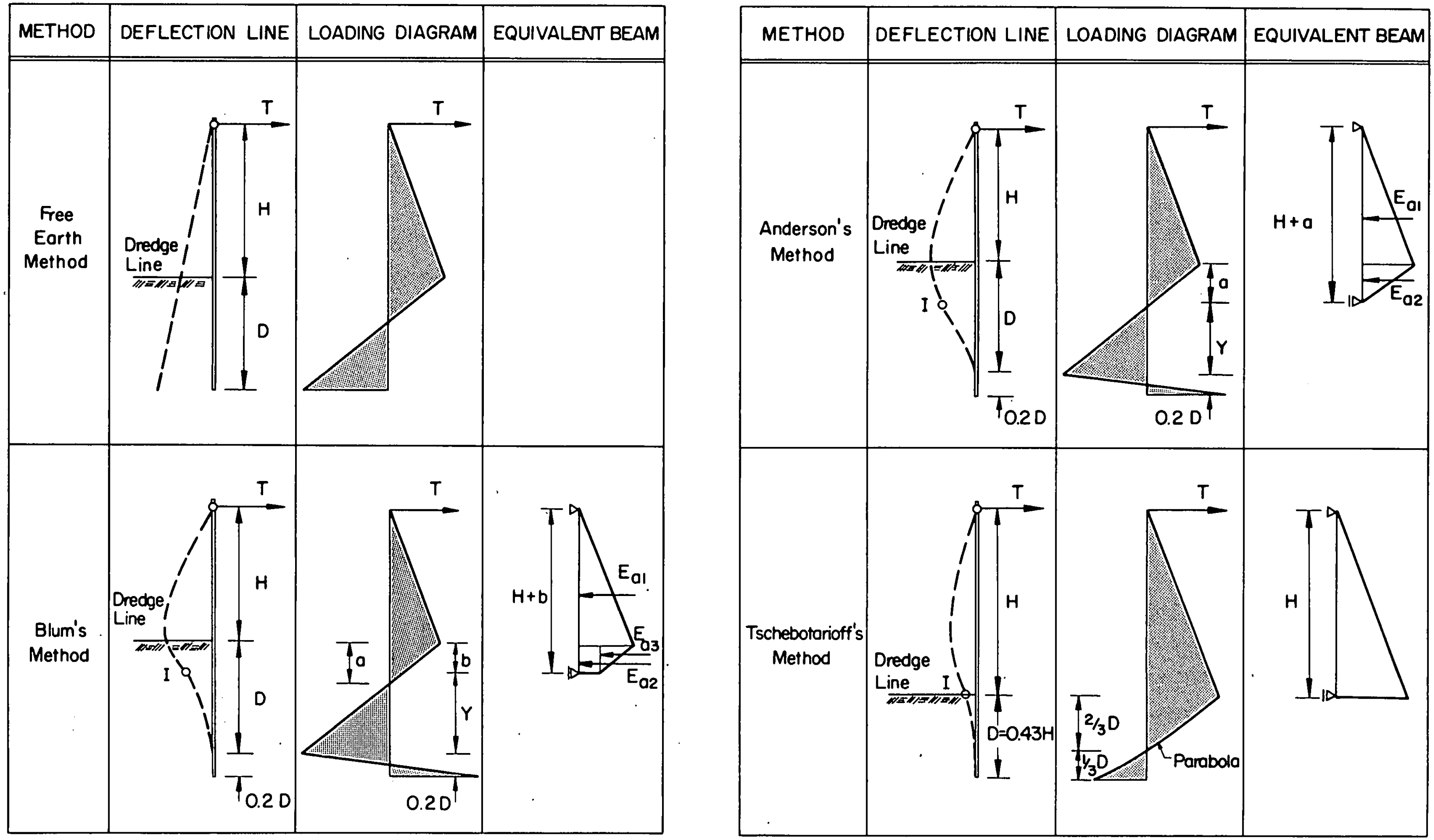

Fig. 1 Assumptions of Free-Earth, Blum, Anderson and Tschebotarioff Methods

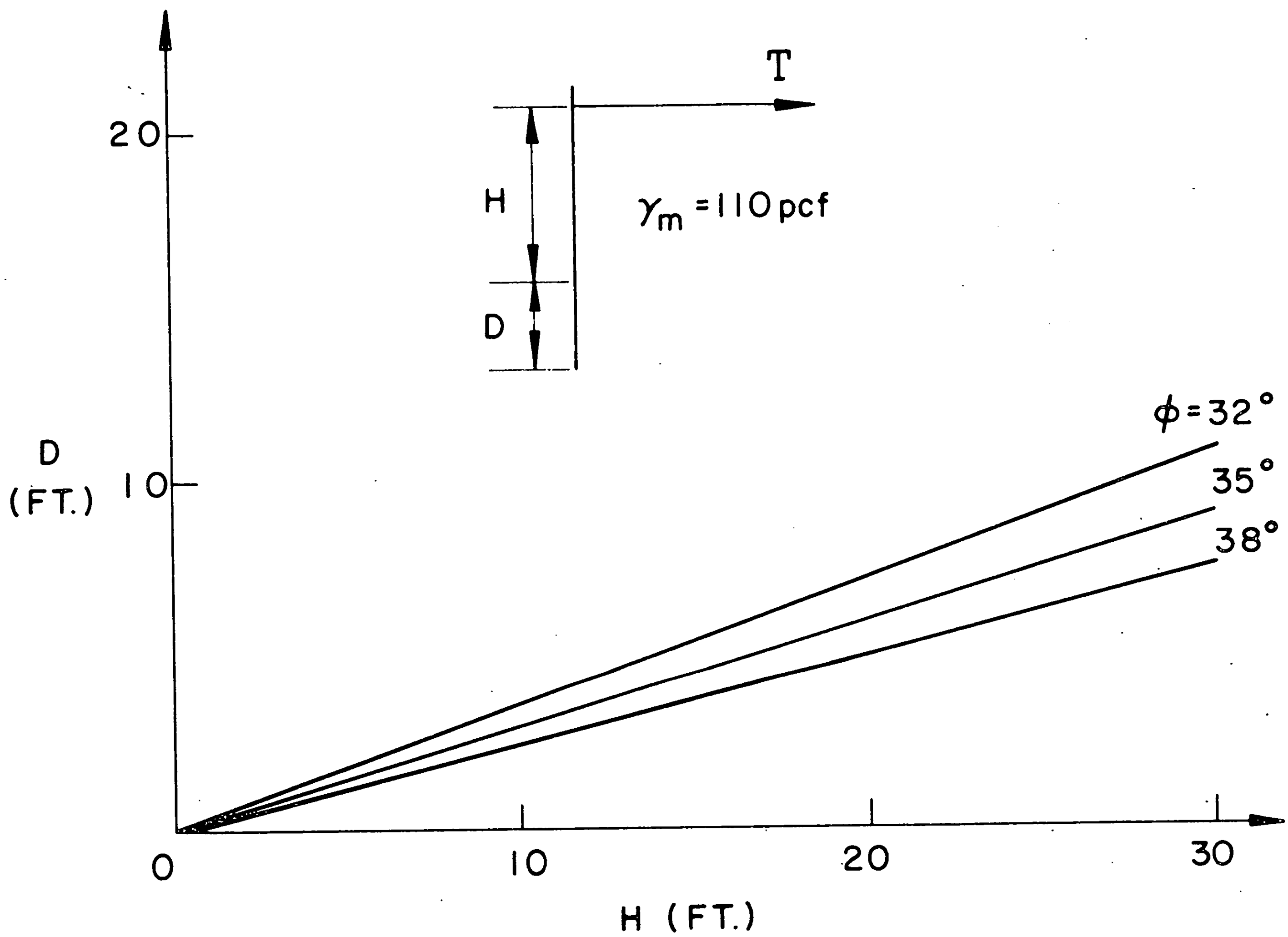

Fig. 2 Free-Earth Method: Theoretical Embedment Depth D

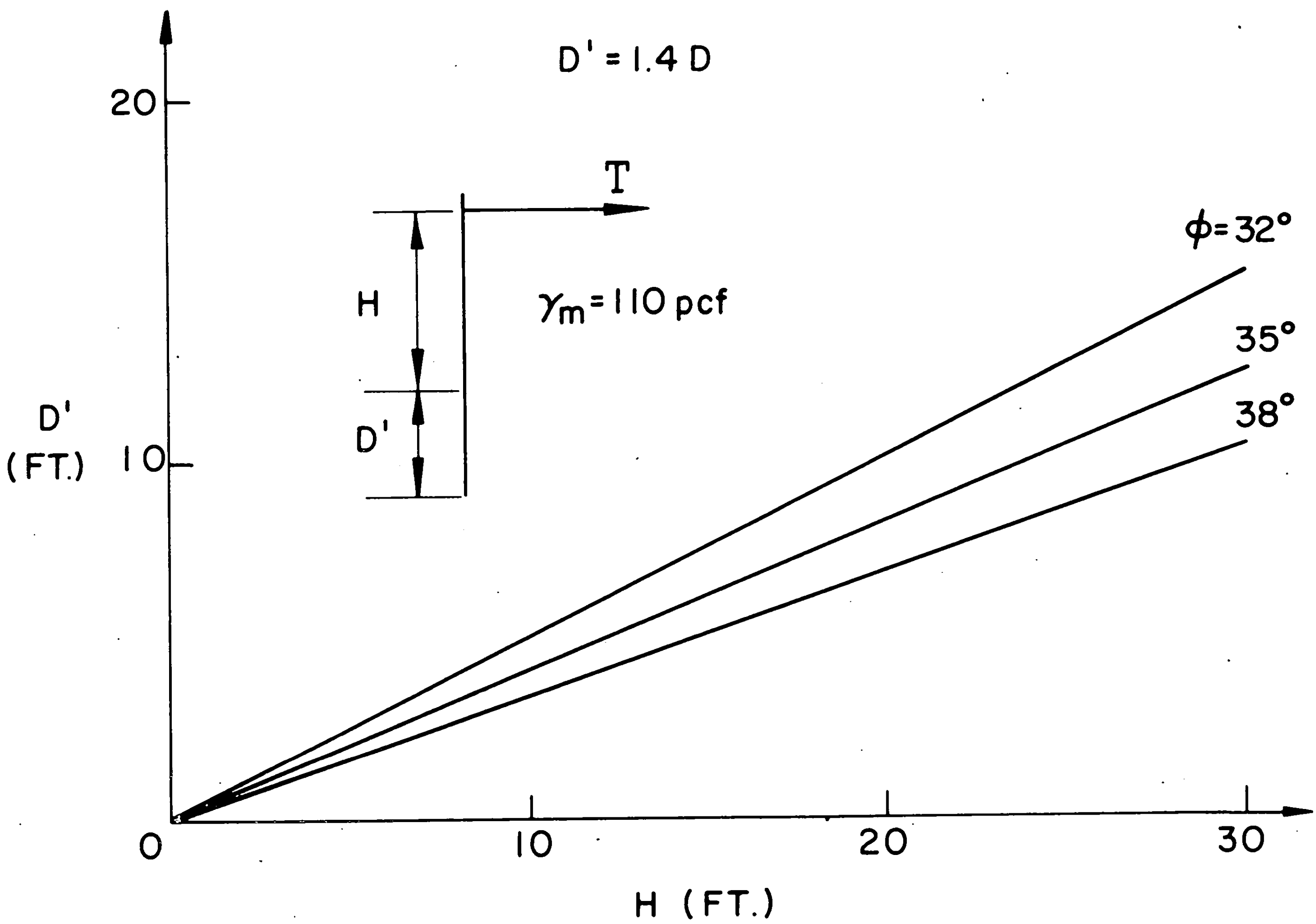

Fig. 3 Free-Earth Method: Recommended Embedment Depth D'

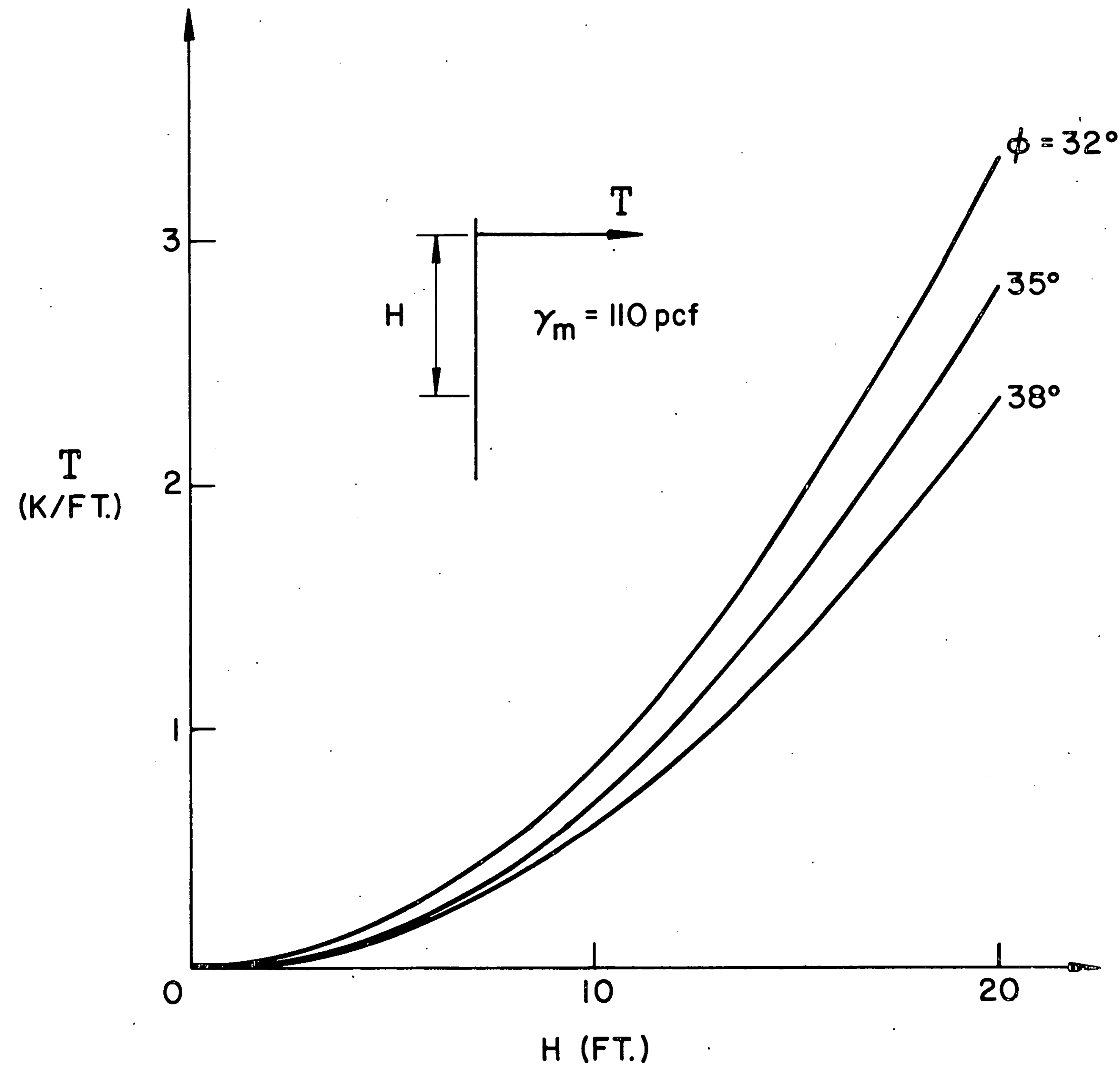

Fig. 4 Free-Earth Method: Tie Rod Force T

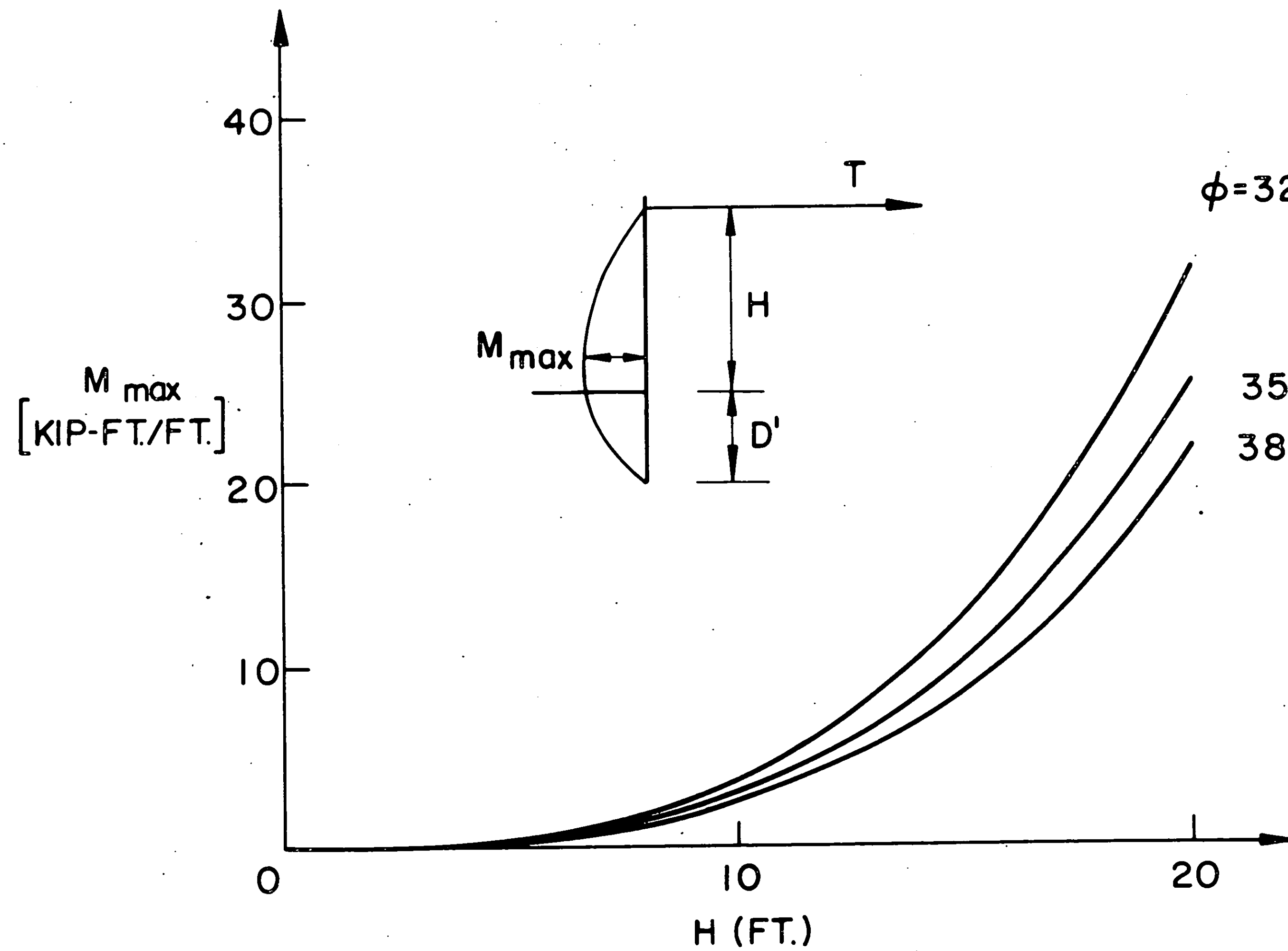

Fig. 5 Free-Earth Method: Maximum Bending Moment M_{max}

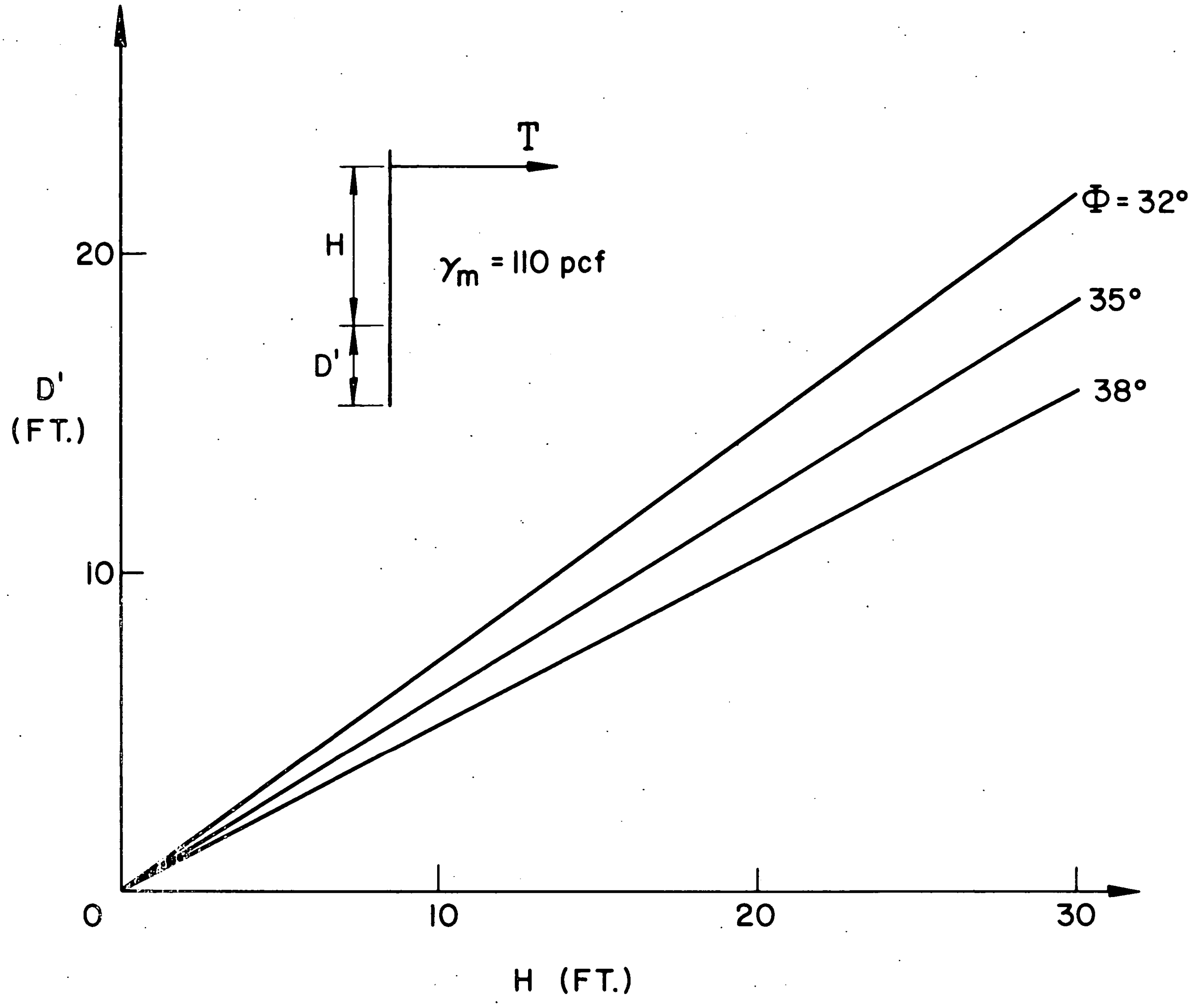

Fig. 6 Blum Method: Embedment Depth D'

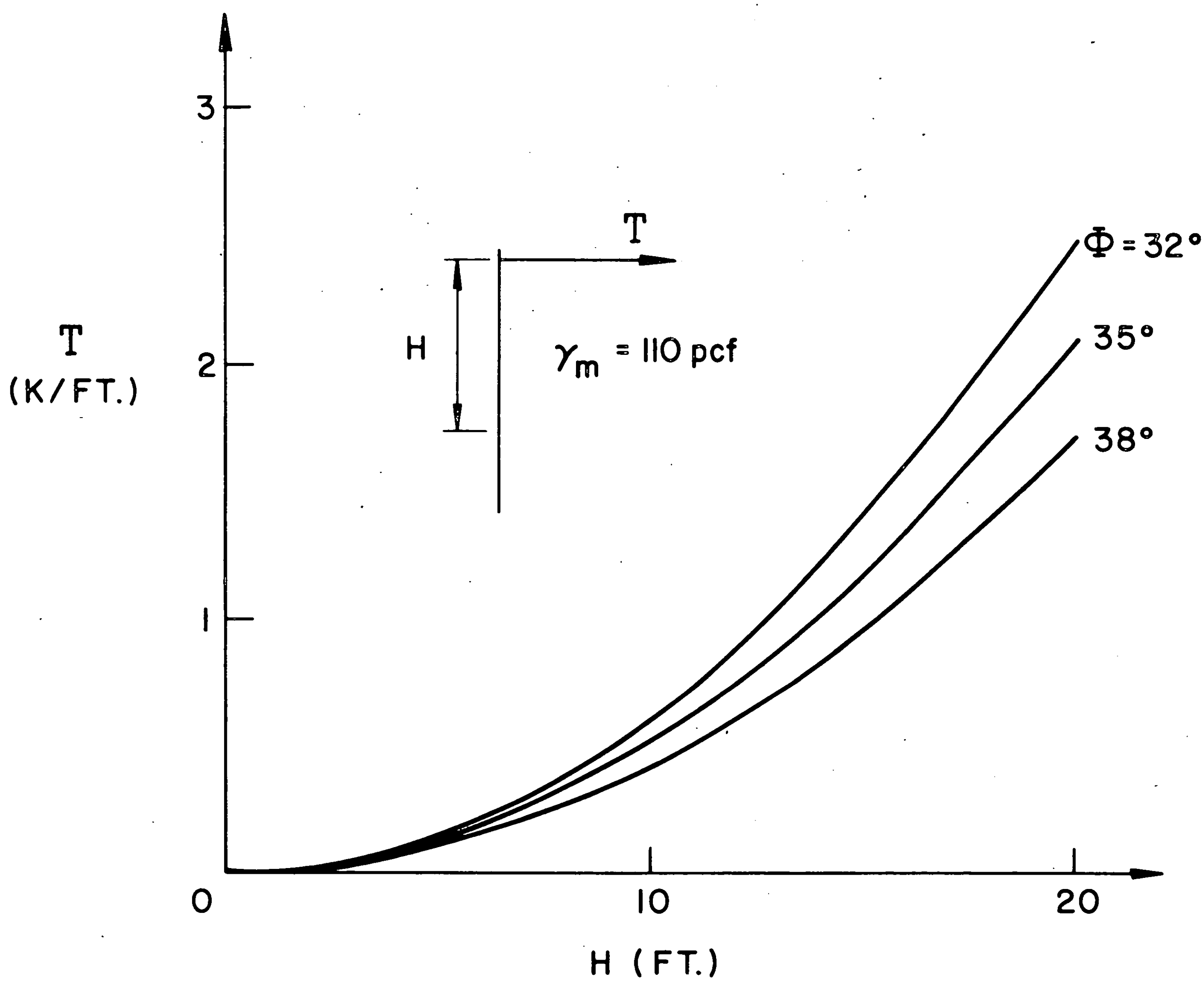

Fig. 7 Blum Method: Tie Rod Force T

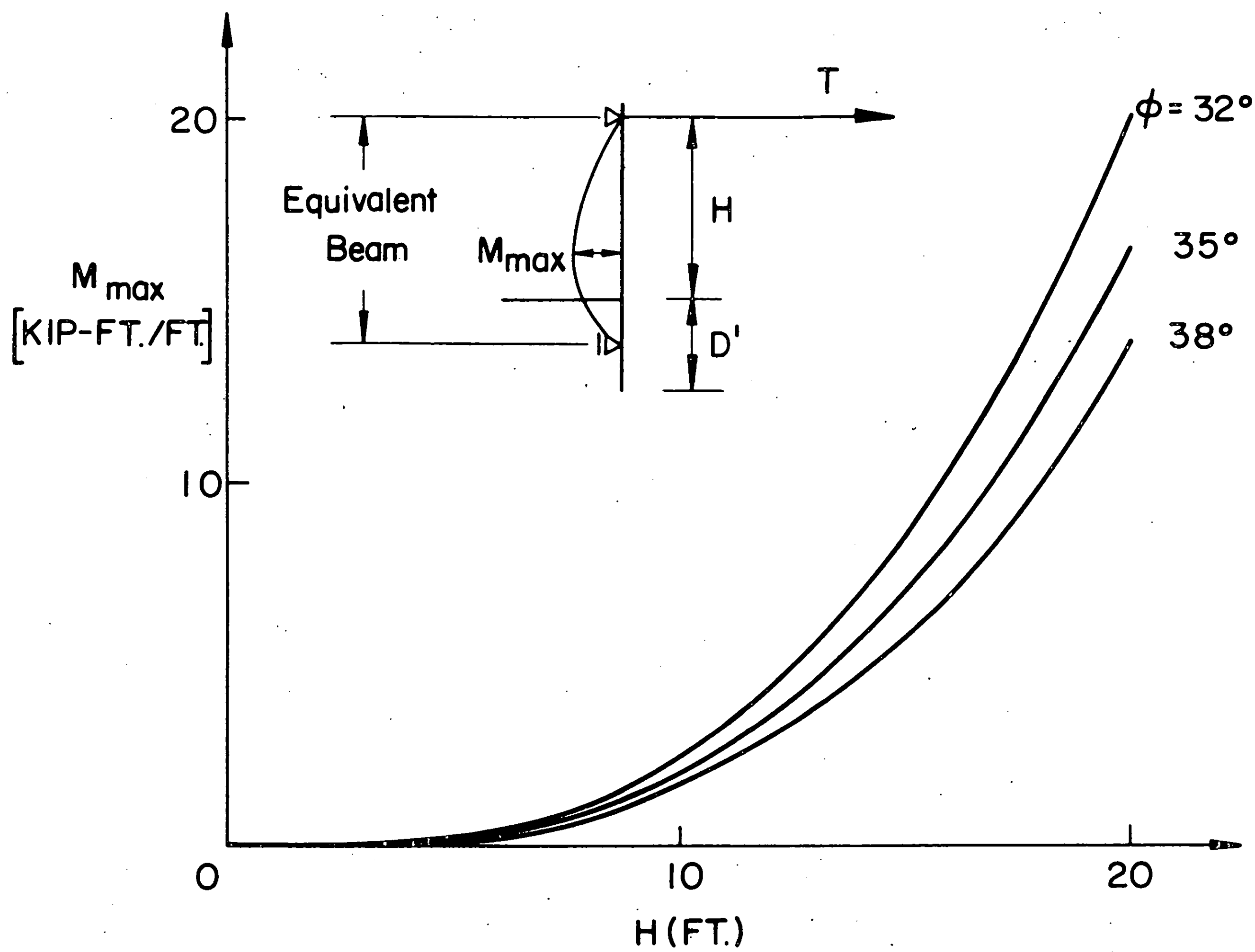

Fig. 8 Blum Method: Maximum Bending Moment M_{max}

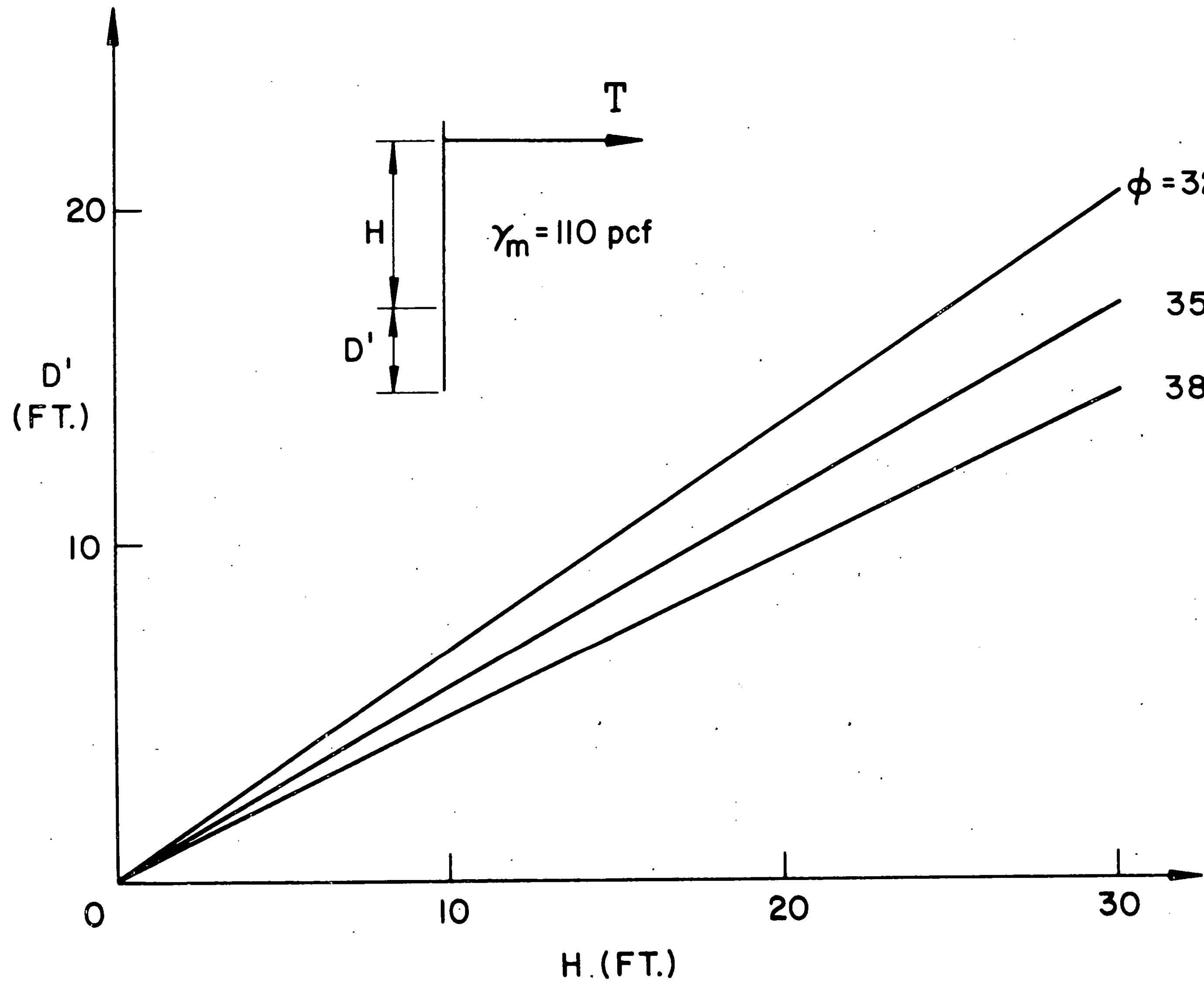

Fig. 9 Anderson Method: Embedment Depth D'

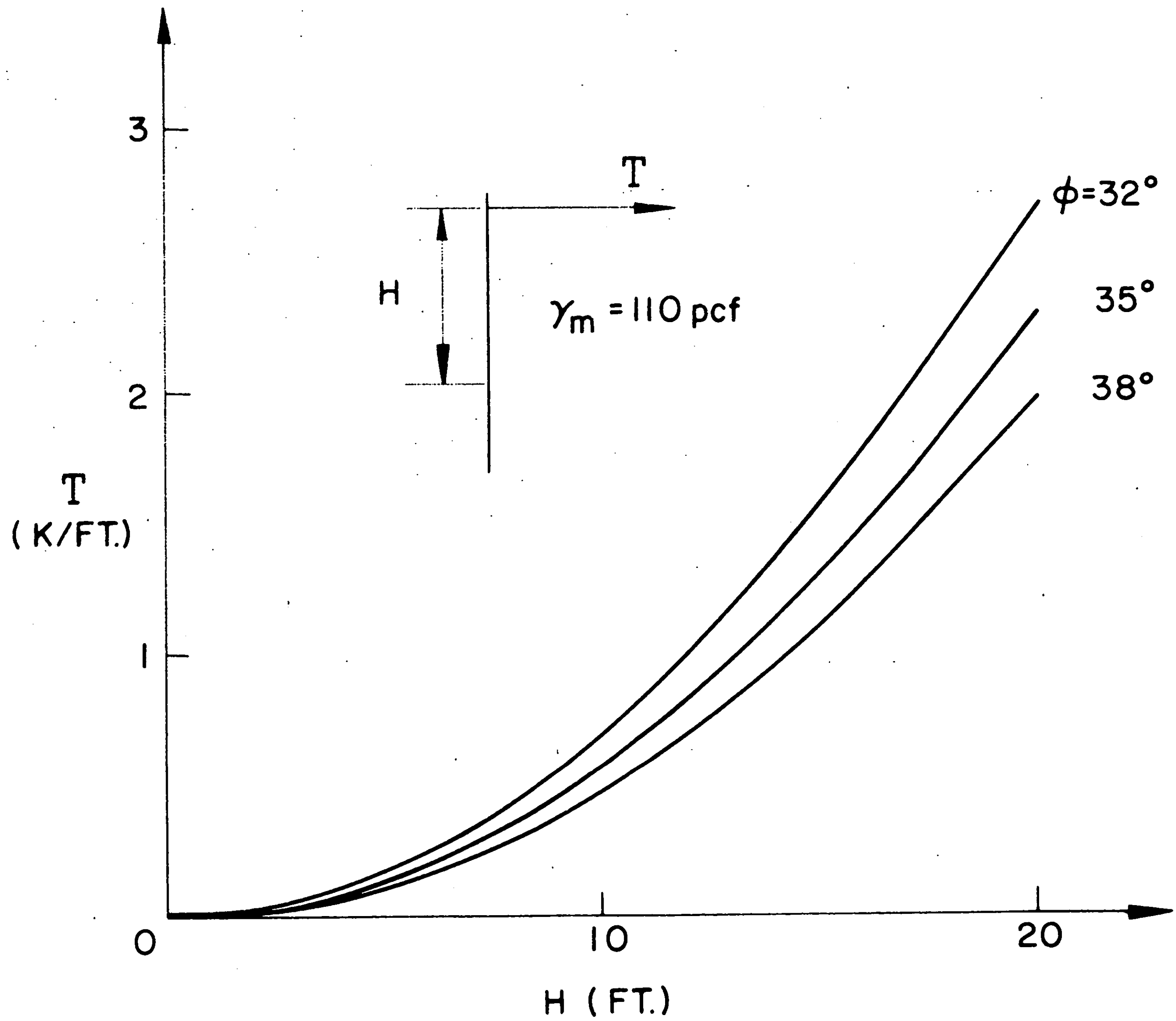

Fig. 10 Anderson Method: Tie Rod Force T

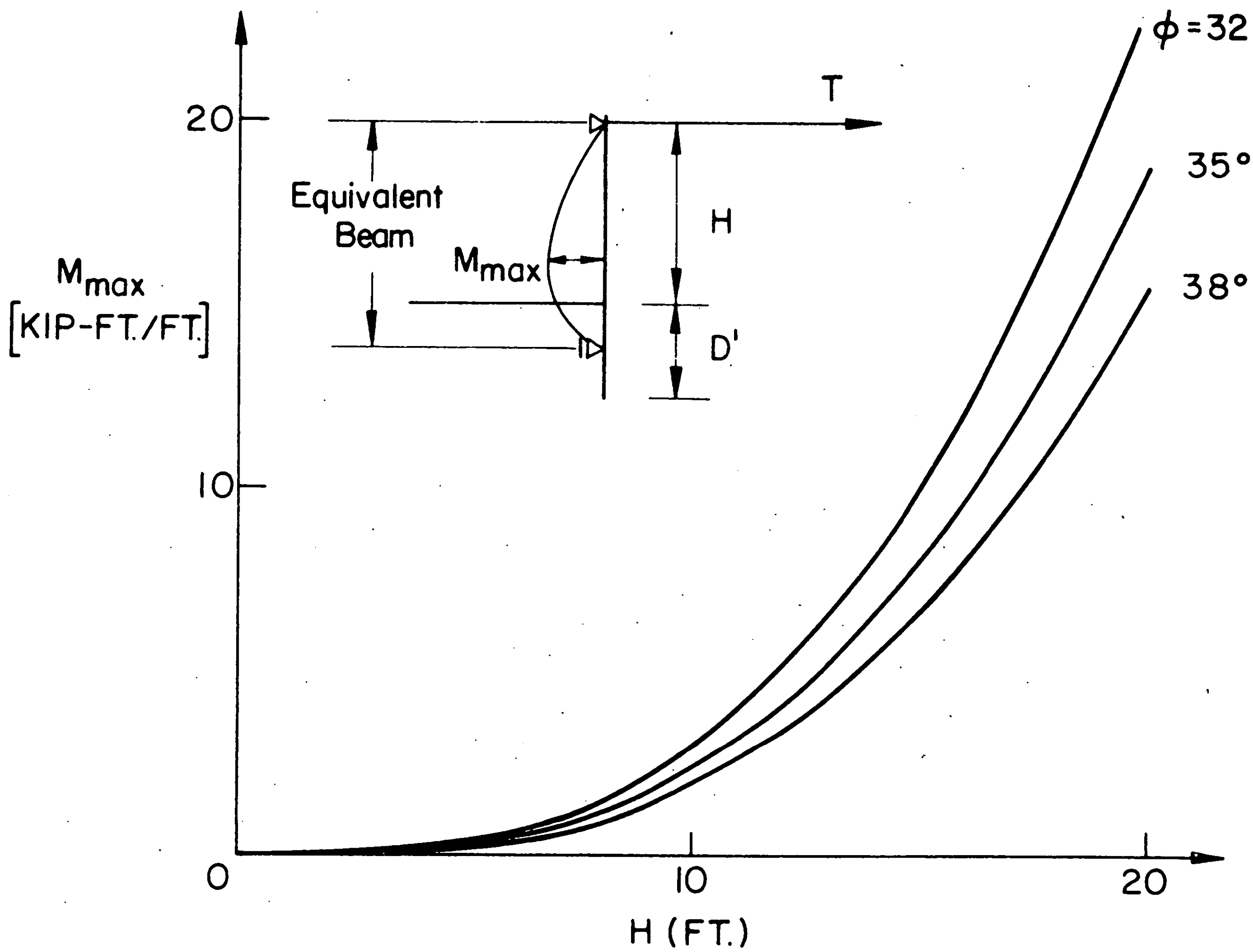

Fig. 11 Anderson Method: Maximum Bending Moment M_{max}

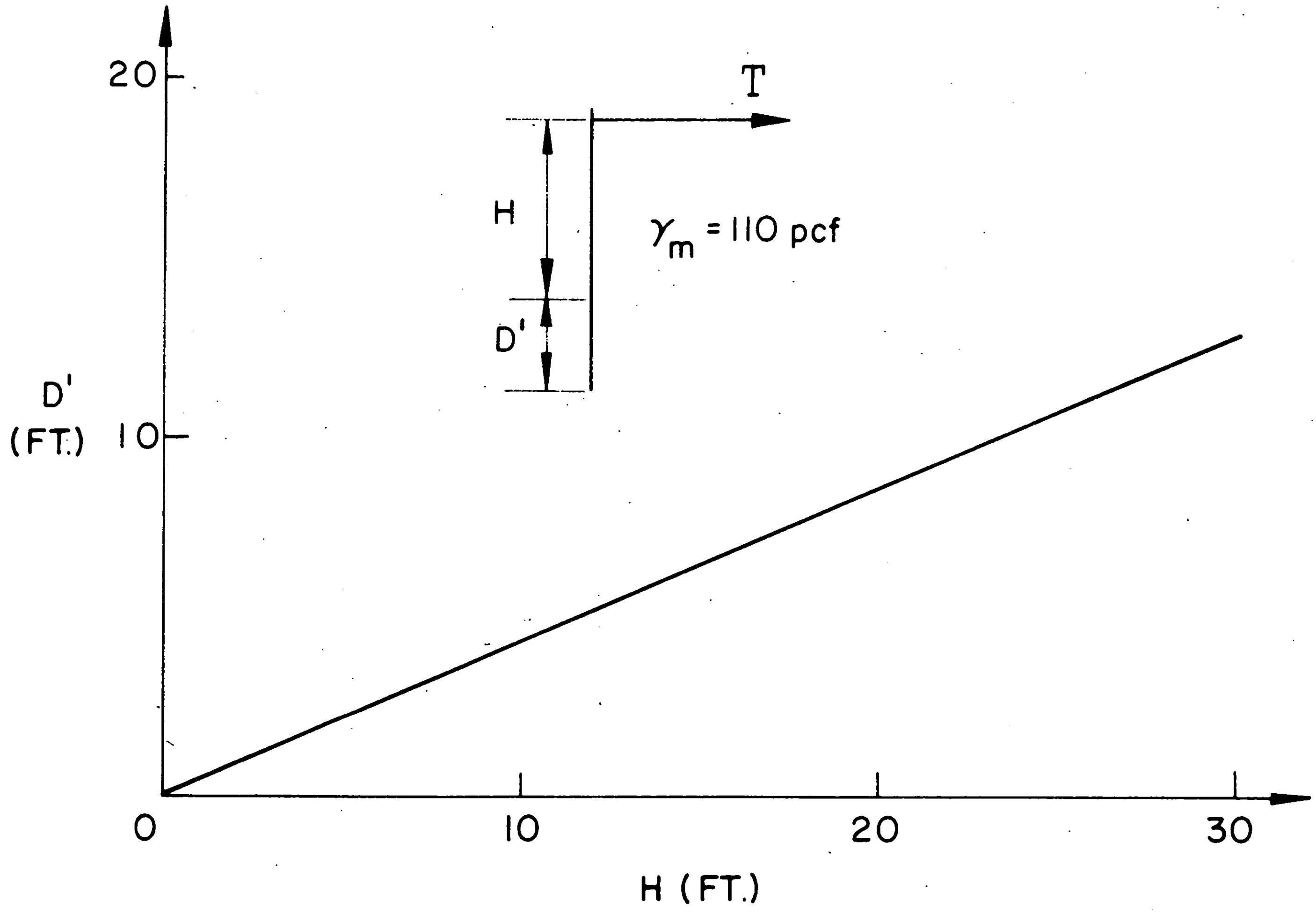

Fig. 12 Tschebotarioff Method: Embedment Depth D'

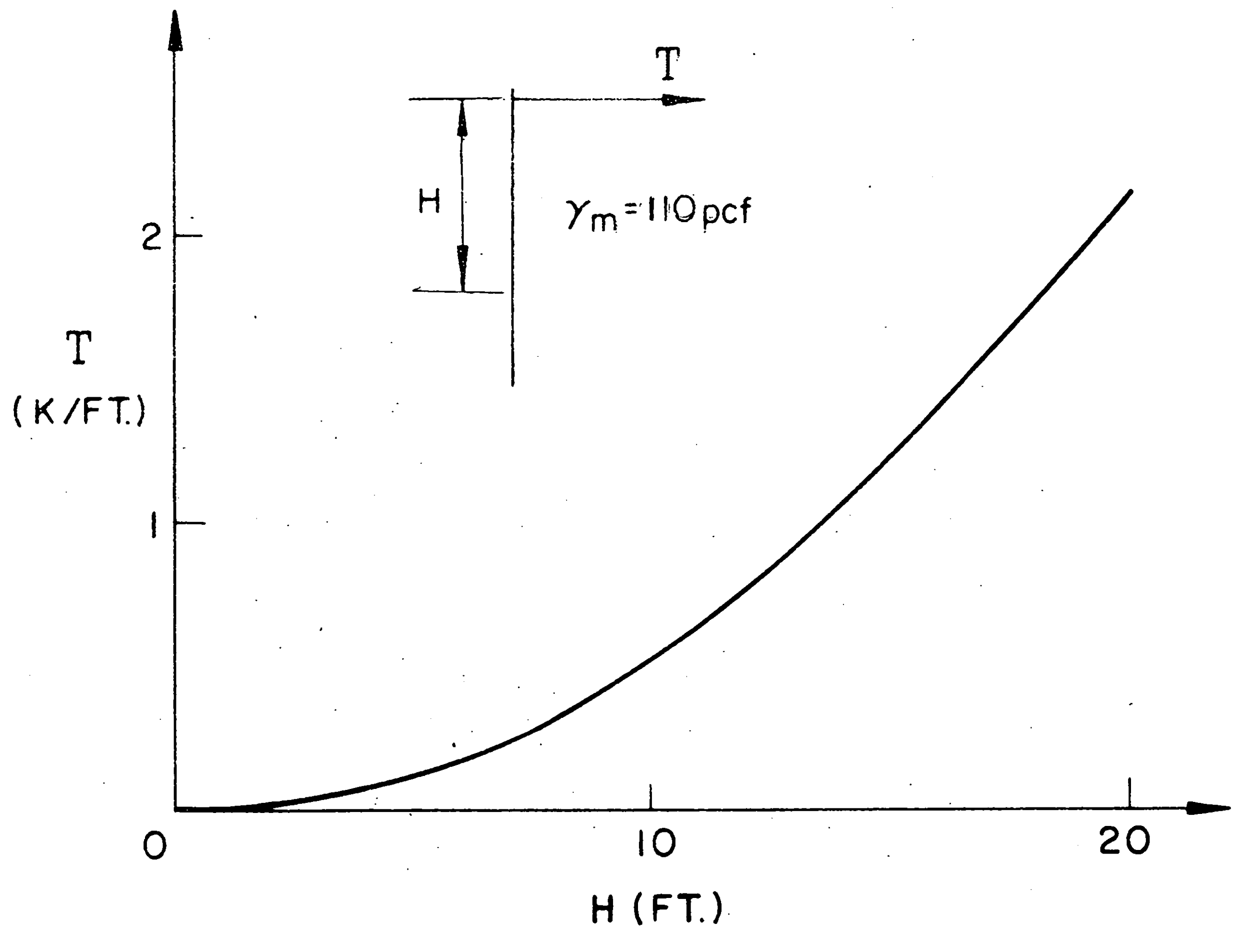

Fig. 13 Tschebotarioff Method: Tie Rod Force T

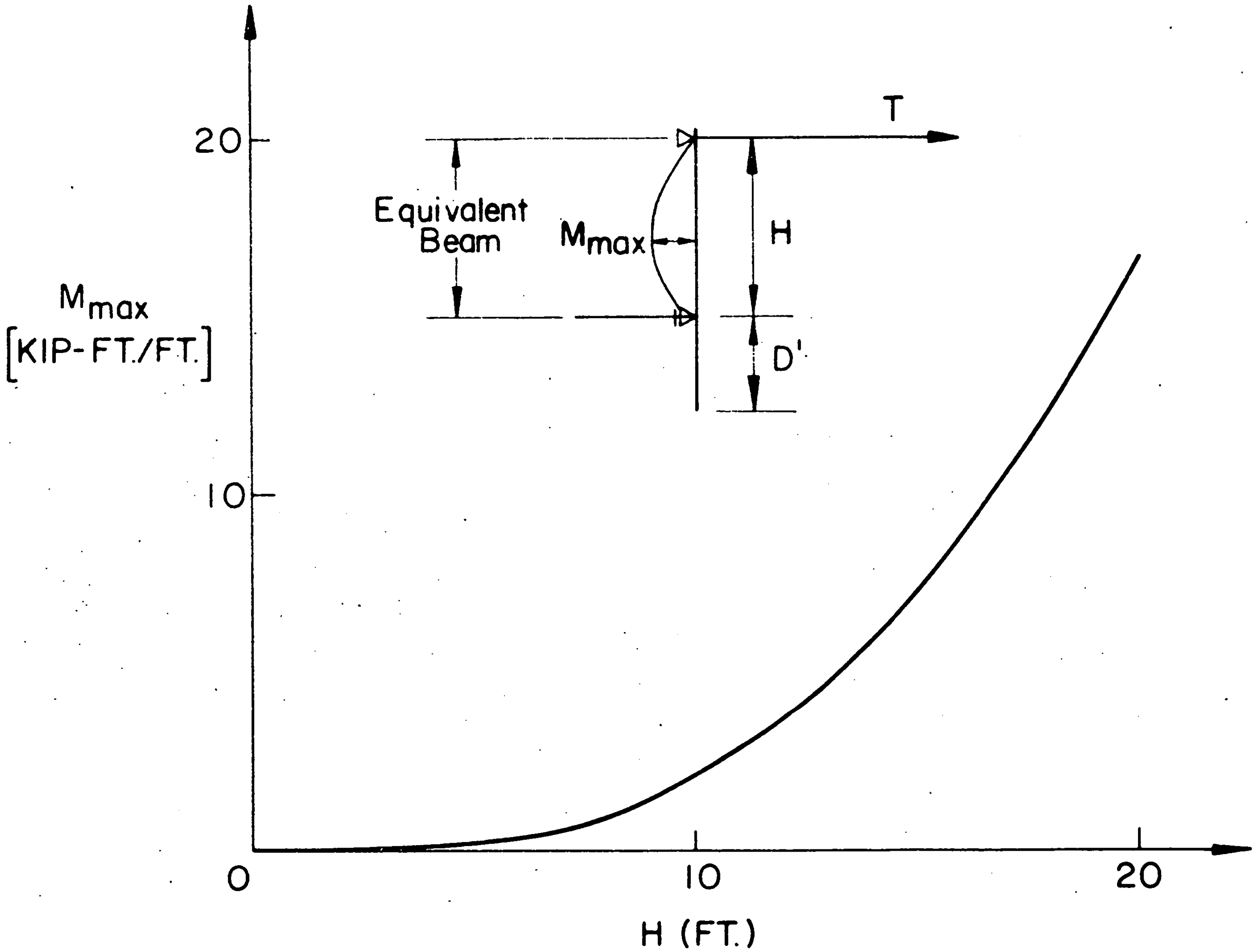

Fig. 14 Tschebotarioff Method: Maximum Bending Moment M_{max}

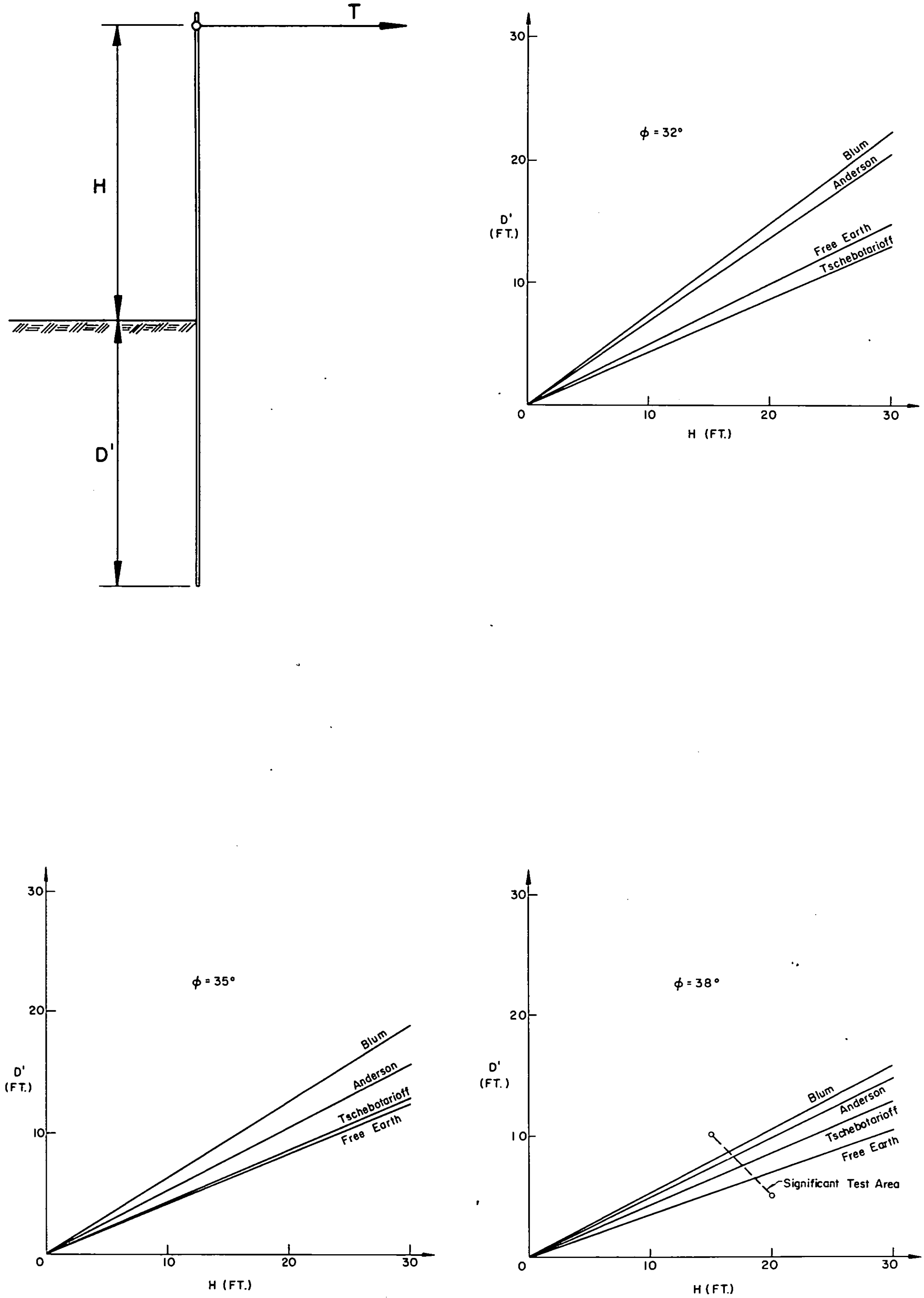

Fig. 15 Comparison of Embedment Depth for ϕ = 32°, 35°, and 38°

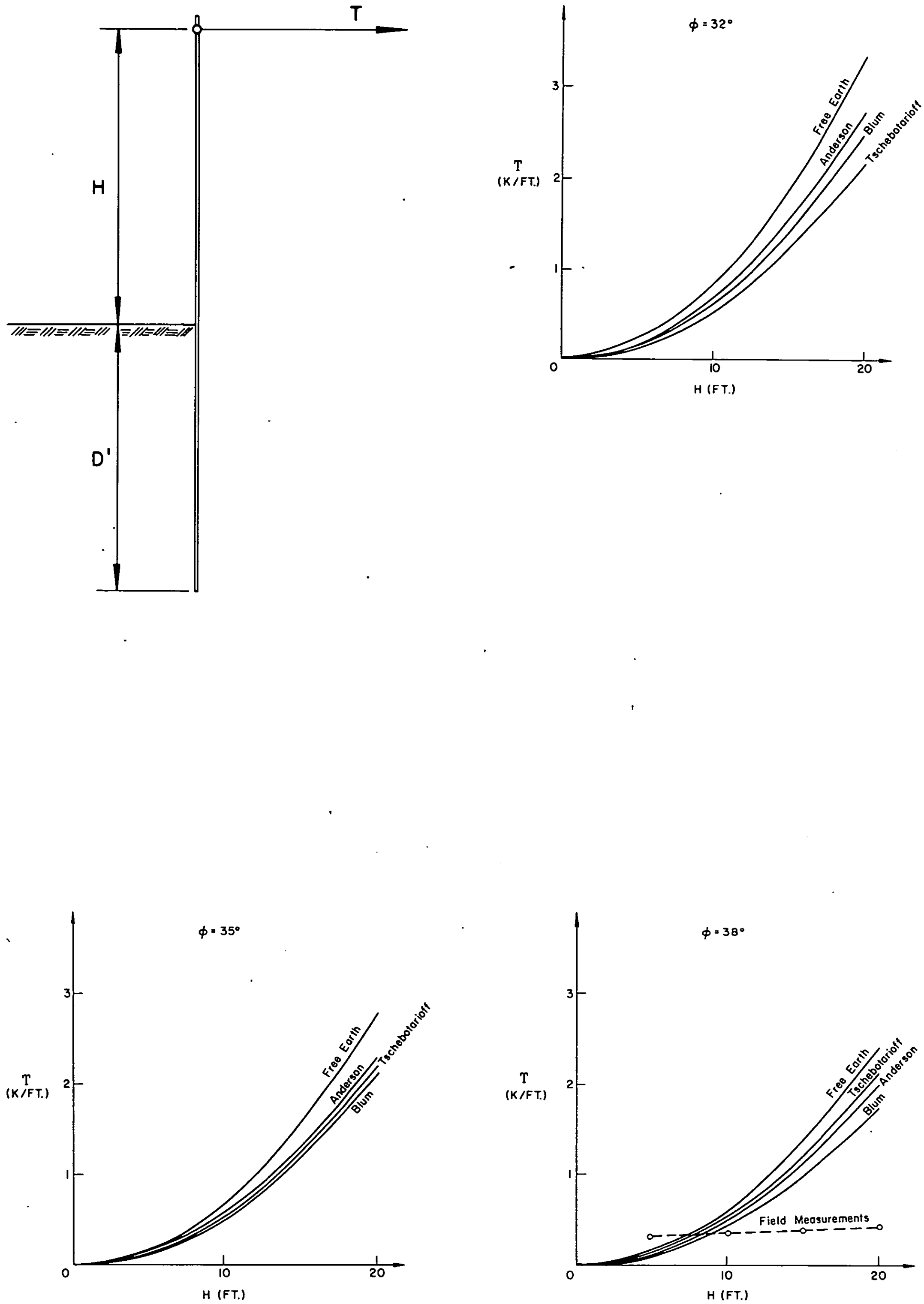

Fig. 16 Comparison of Tie Rod for $\phi = 32°, 35°,$ and $38°$

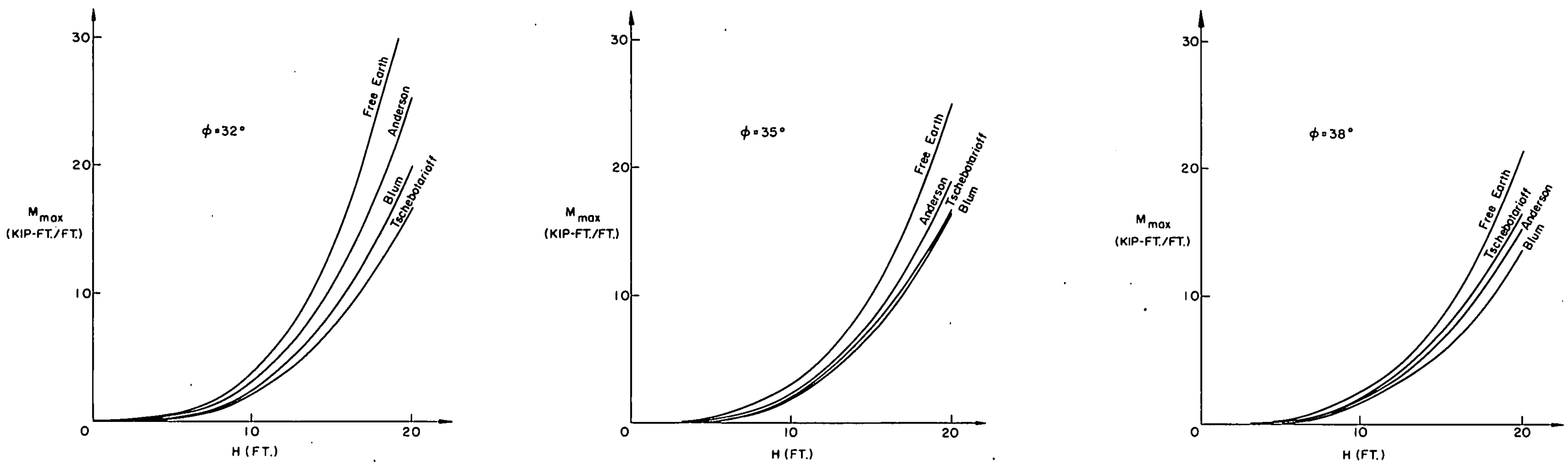

Fig. 17 Comparison of Maximum Bending Moment for $\phi = 32°$, $35°$, and $38°$

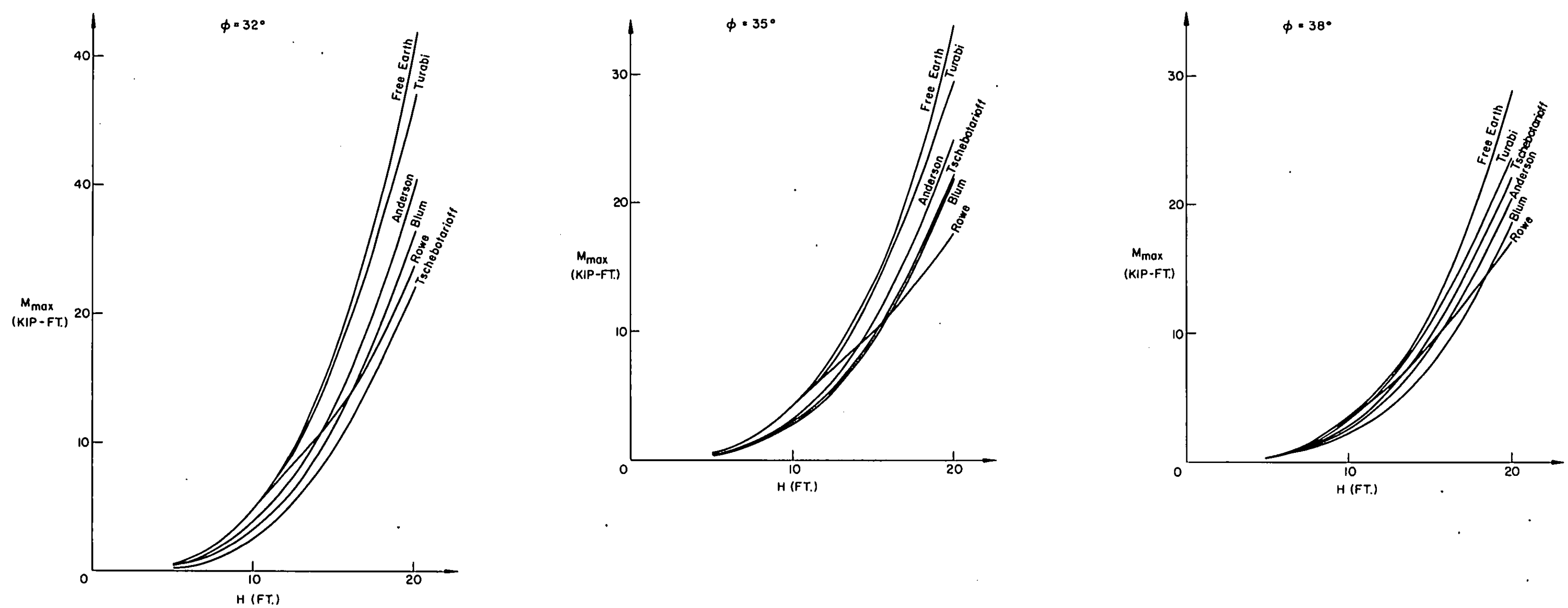

Fig. 18 Comparison of Maximum Bending Moment for DP-2 Sheet Pile

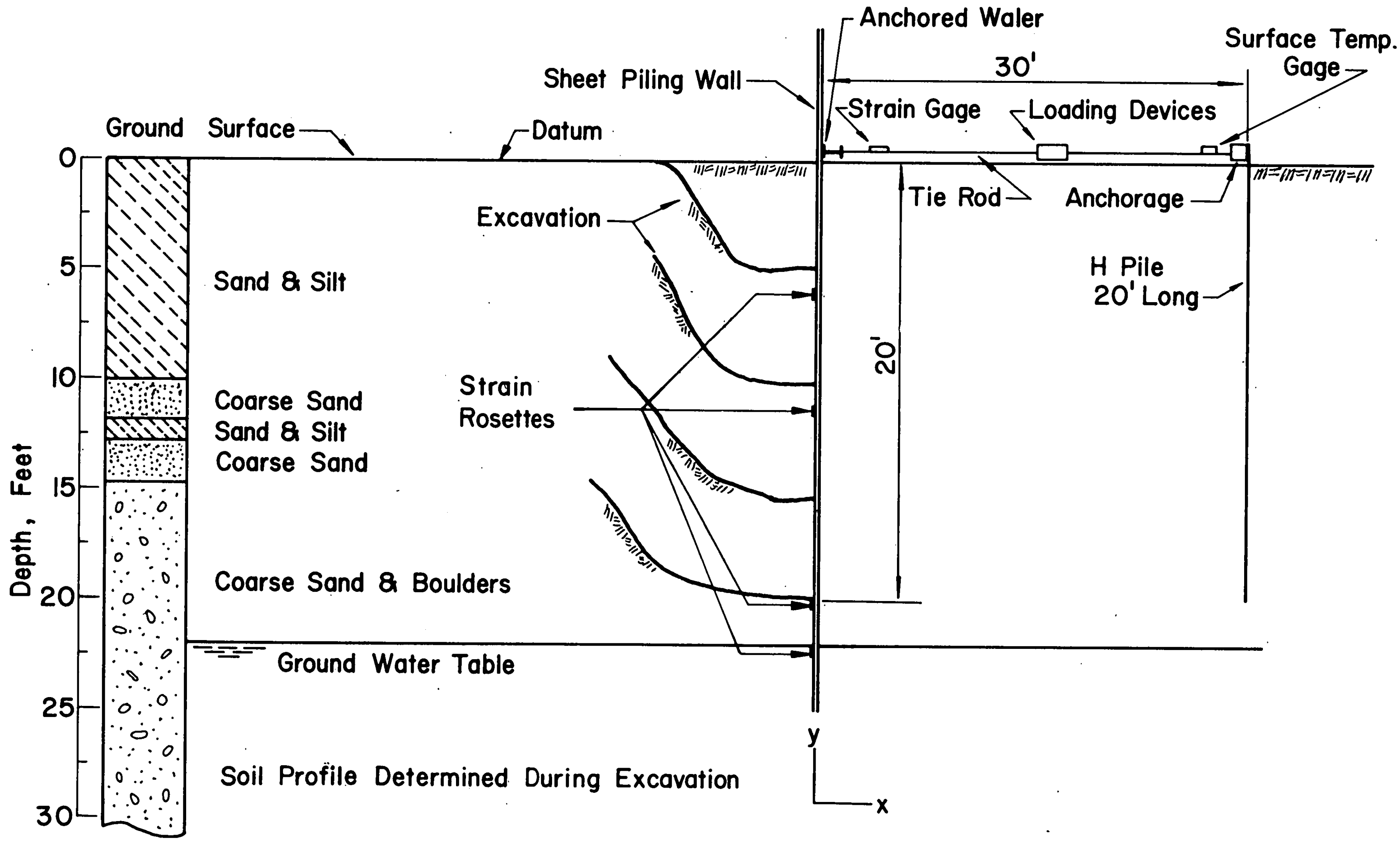

Fig. 19 Cross-Section of Sheet Pile Wall

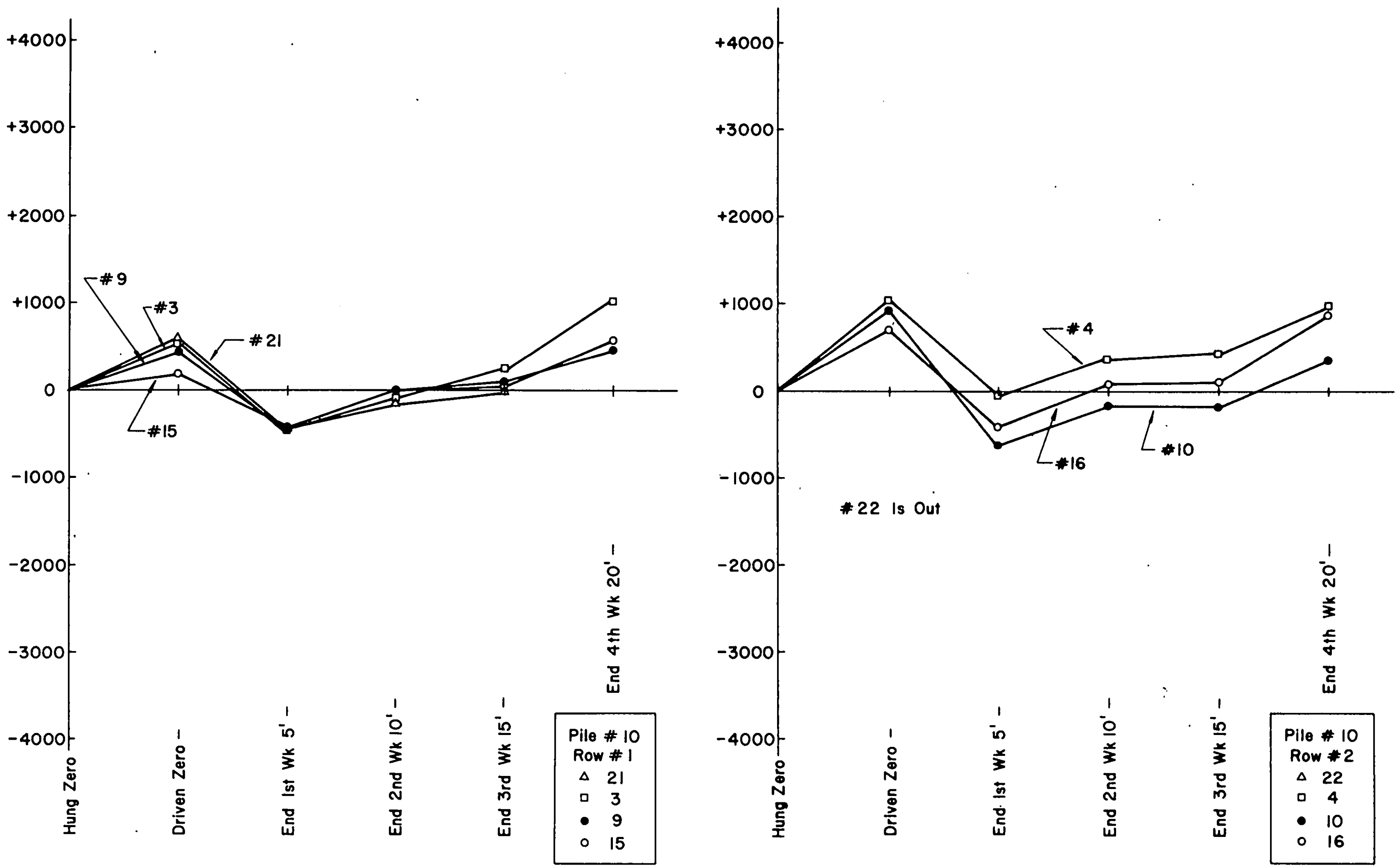

Fig. 21 Strain Distribution from Field Measurements - Pile #10

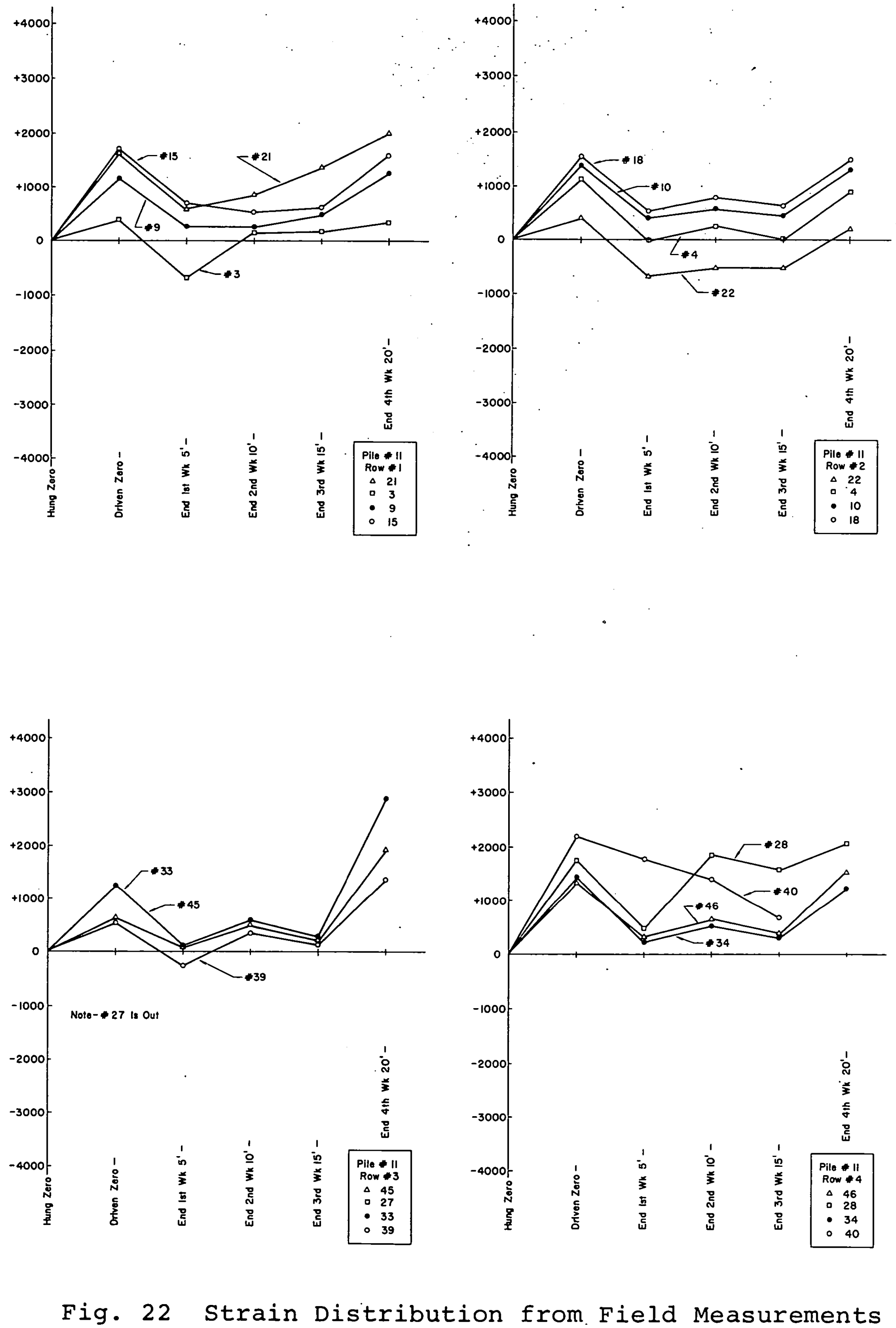

Fig. 22 Strain Distribution from Field Measurements - Pile #11

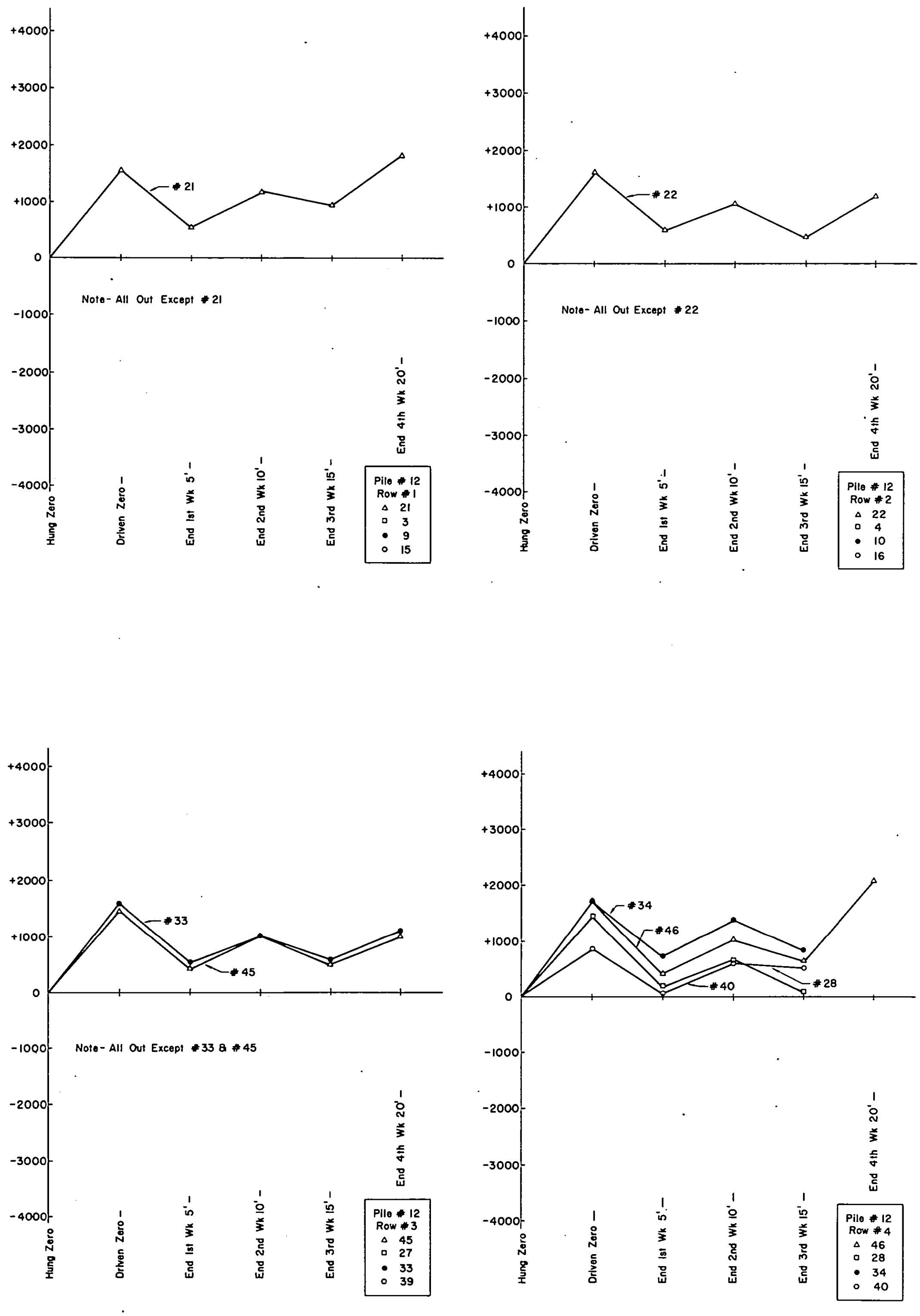

Fig. 23 Strain Distribution from Field Measurements - Pile #12

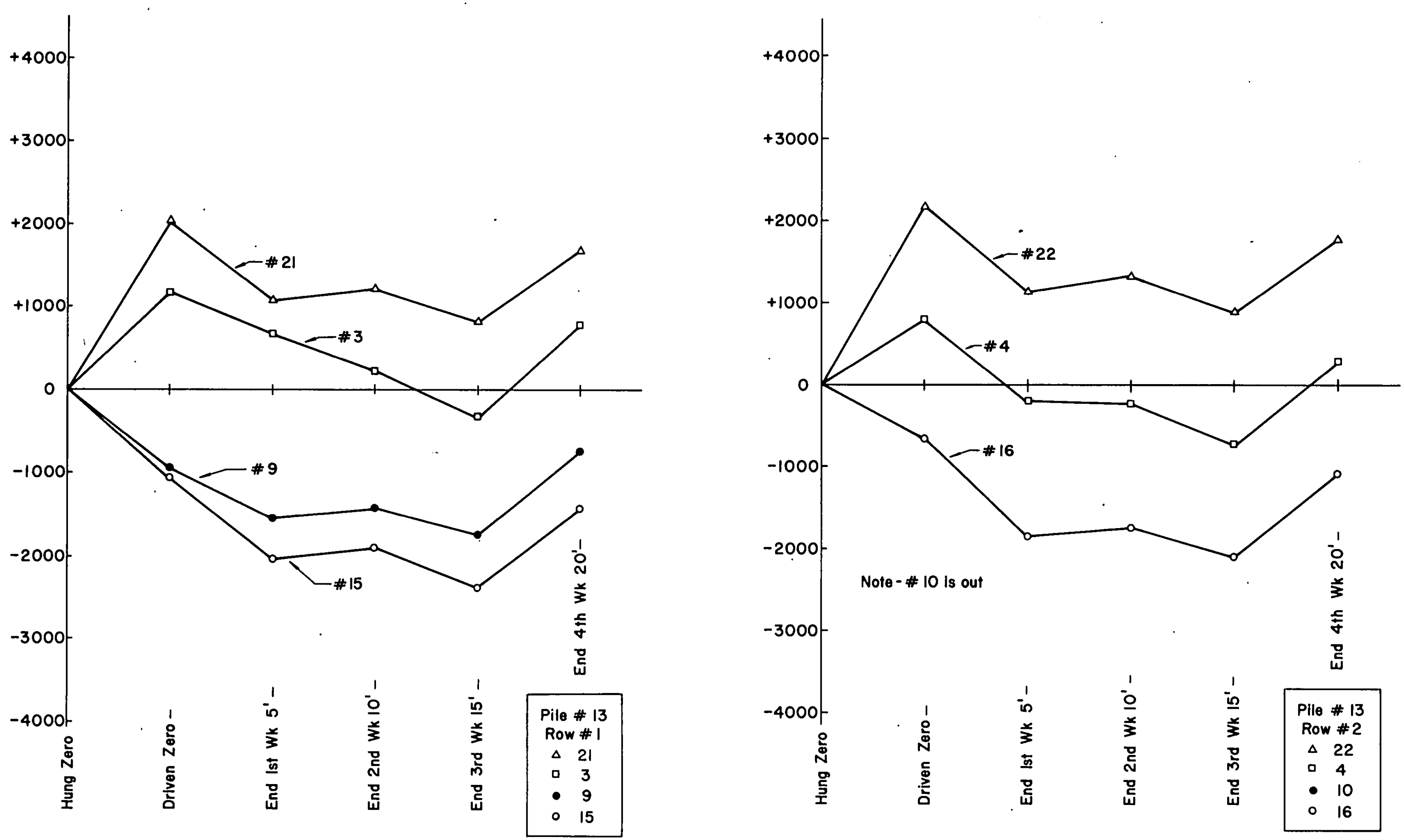

Fig. 24 Strain Distribution from Field Measurements - Pile #13

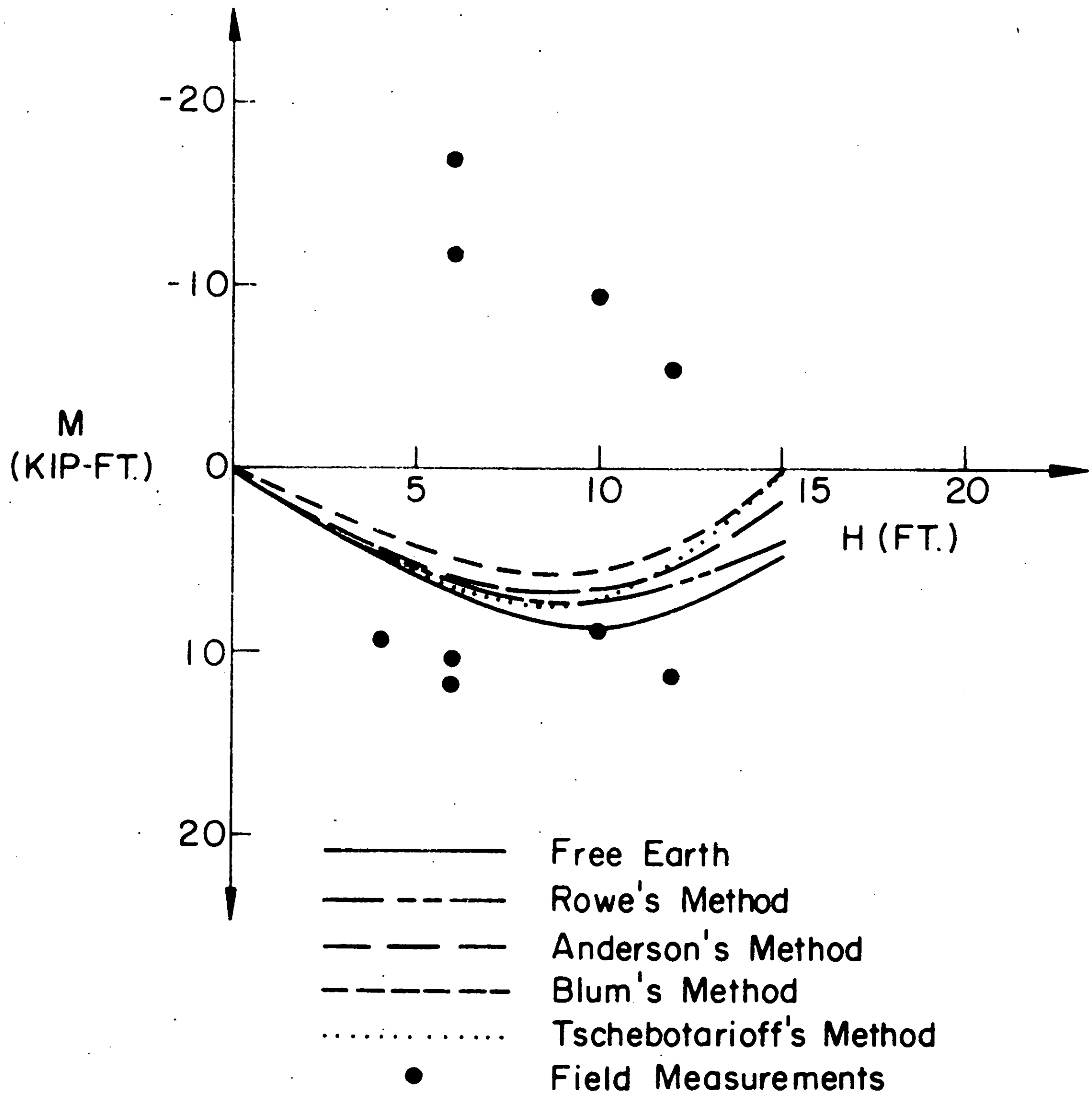

Fig. 25 Comparison of Theoretical Moment Curves with Field Measurements for a wall 15 ft. high

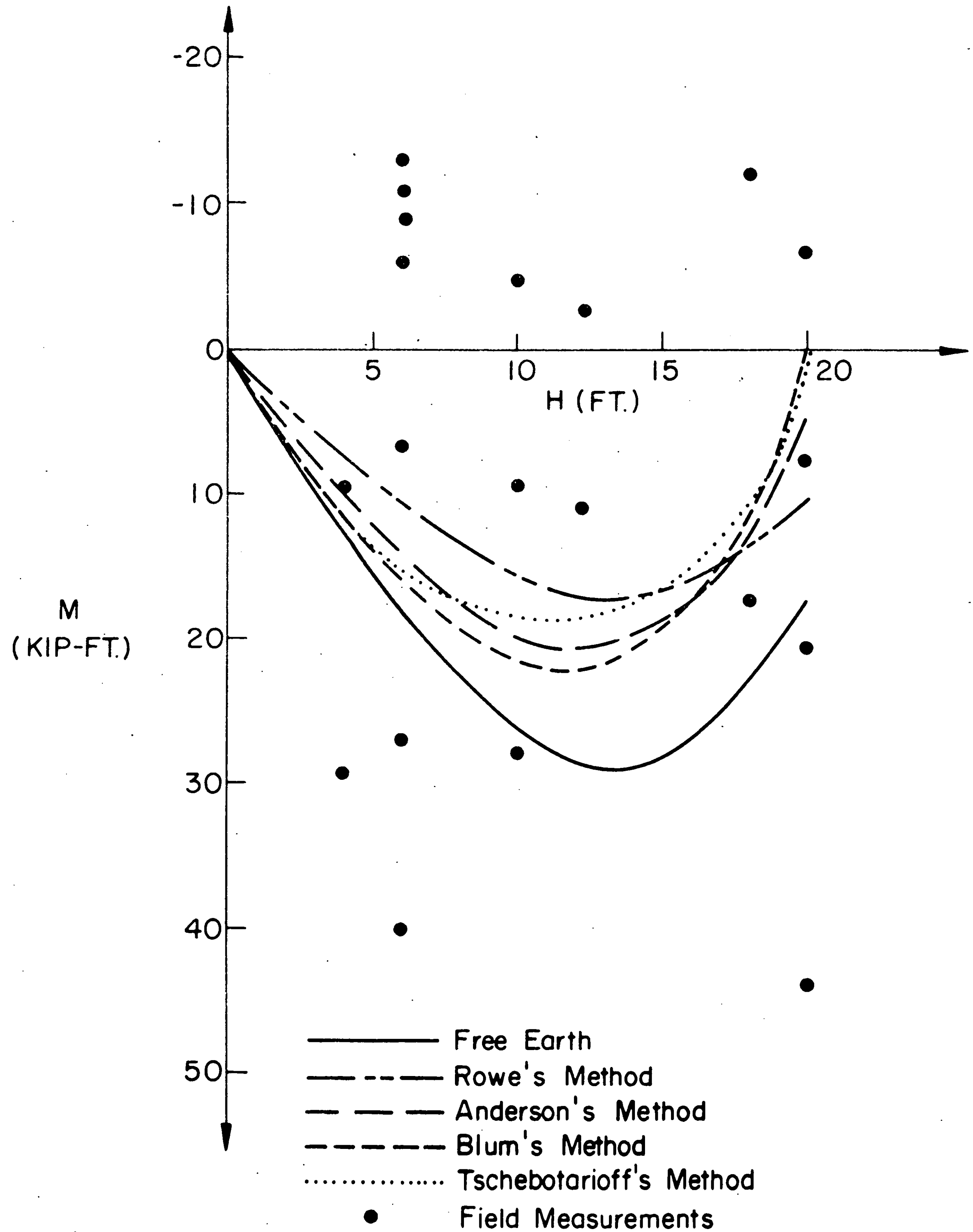

Fig. 26 Comparison of Theoretical Moment Curves with Field Measurements for a wall 20 ft. high

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10. APPENDIX

Type of soil	N	D_r	ϕ	
			Peck	Meyerhof
Very loose sand	< 4	< 0.2	< 29	< 30
Loose sand	4-10	0.2-0.4	29-30	30-35
Medium sand	10-30	0.4-0.6	30-36	35-40
Dense sand	30-50	0.6-0.8	36-41	40-45
Very dense sand	> 50	> 0.8	> 41	> 45

Relationship between N, D_r, and Φ after Peck (1953) and Meyerhof (1956)

Conversion to Other Units

$1\ kp = 0.001\ Mp = 0.0022\ kip = 2.20\ lb.$

$1\ cm = 0.01\ m = 0.033\ ft. = 0.394\ in.$

$1\ kp\ cm^{-2} = 10\ Mp\ m^{-2} = 2.05\ kip\ ft.^{-2} = 14.22\ lb\ in.^{-2}$

$1\ kp\ cm^{-3} = 1000\ M_p\ m^{-3} = 62.43\ kip\ ft.^{-3} = 36.13\ lb.\ in.^{-3}$

$1\ lb. = 0.001\ kip = 0.454\ kp = 4.54 \times 10^{-4}\ Mp$

$1\ in. = 0.083\ ft. = 2.54\ cm = 0.0254\ m$

$1\ lb.\ in.^{-2} = 0.144\ kip\ ft.^{-2} = 0.070\ kp\ cm^{-2} = 0.703\ Mp\ m^{-2}$

$1\ lb.\ in.^{-3} = 1.728\ kip\ ft.^{-3} = 0.028\ kp\ cm^{-3} = 27.68\ Mp\ m^{-3}$