EXAMPLE NO 4 USER INPUT P-Y CURVES

UNITS--ENGL

****
PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE
****

INPUT INFORMATION
****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 720.00 IN
MODULUS OF ELASTICITY OF PILE = .290E+05 KIP/IN**2
2 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00			
	16.000	.105E+04	.359E+02
180.00			
	16.000	.720E+03	.239E+02
720.00			

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = 60.00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER RESPONSE IS DEFINED BY INPUT P-Y CURVES
X AT THE TOP OF THE LAYER = 60.00 IN

```
X AT THE BOTTOM OF THE LAYER      =      720.00 IN
VARIATION OF SOIL MODULUS, k      =    .500E+02 LBS/IN**3


INPUT P-Y CURVES        7 CURVES,  6 POINTS ON EACH

          X,IN              Y,IN            P,LBS/IN
         60.00               .00                 .00
                             .20               66.10
                             .40               83.20
                             .80              105.00
                            1.20              120.00
                            6.00                 .00

          X,IN              Y,IN            P,LBS/IN
         76.00               .00                 .00
                             .20               79.80
                             .40              100.00
                             .80              127.00
                            1.20              145.00
                            6.00               15.00

          X,IN              Y,IN            P,LBS/IN
         92.00               .00                 .00
                             .20               93.30
                             .40              117.00
                             .80              148.00
                            1.20              169.00
                            6.00               34.00

          X,IN              Y,IN            P,LBS/IN
        108.00               .00                 .00
                             .20              107.00
                             .40              135.00
                             .80              170.00
                            1.20              194.00
                            6.00               61.00

          X,IN              Y,IN            P,LBS/IN
        140.00               .00                 .00
                             .20              134.00
                             .40              169.00
                             .80              213.00
                            1.20              243.00
                            6.00              123.00

          X,IN              Y,IN            P,LBS/IN
        188.00               .00                 .00
                             .20              175.00
                             .40              221.00
                             .80              278.00
```

	1.20	318.00
	6.00	264.00

X,IN	Y,IN	P,LBS/IN
214.00	.00	.00
	.20	198.00
	.40	250.00
	.80	315.00
	1.20	360.00
	6.00	360.00

```
FINITE DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                =       120
    TOLERANCE ON DETERMINATION OF DEFLECTIONS                =   .100E-03 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS   =       100
    MAXIMUM ALLOWABLE DEFLECTION                             =    .10E+03 IN

INPUT CODES
    OUTPT  =  1
    KCYCL  =  1
    KBC    =  1
    KPYOP  =  0
    INC    =  3

EXAMPLE NO 4 USER INPUT P-Y CURVES

UNITS--ENGL
```

O U T P U T   I N F O R M A T I O N
****

---------- *** ----------

```
PILE LOADING CONDITION

    LATERAL LOAD AT PILE HEAD       =   .500E+01 KIP
    APPLIED MOMENT AT PILE HEAD     =   .000E+00 IN-KIP
    AXIAL LOAD AT PILE HEAD         =   .100E+03 KIP
```

X	DEFLECTION	MOMENT	TOTAL STRESS	SHEAR	SOIL RESIST	FLEXURAL RIGIDITY
IN	IN	IN-KIP	LBS/IN**2	KIP	LBS/IN	KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.453E+00	.000E+00	.279E+04	.532E+01	.000E+00	.305E+08
18.00	.396E+00	.957E+02	.351E+04	.500E+01	.000E+00	.305E+08
36.00	.340E+00	.191E+03	.424E+04	.500E+01	.000E+00	.305E+08
54.00	.286E+00	.287E+03	.497E+04	.500E+01	.000E+00	.305E+08
72.00	.235E+00	.374E+03	.563E+04	.387E+01	.798E+02	.305E+08
90.00	.188E+00	.435E+03	.610E+04	.235E+01	.862E+02	.305E+08
108.00	.146E+00	.468E+03	.635E+04	.869E+00	.780E+02	.305E+08
126.00	.109E+00	.475E+03	.640E+04	-.435E+00	.663E+02	.305E+08
144.00	.762E-01	.460E+03	.629E+04	-.151E+01	.524E+02	.305E+08
162.00	.488E-01	.428E+03	.605E+04	-.231E+01	.373E+02	.305E+08
180.00	.259E-01	.384E+03	.571E+04	-.284E+01	.218E+02	.305E+08
198.00	.795E-02	.331E+03	.787E+04	-.310E+01	.730E+01	.257E+08
216.00	-.488E-02	.276E+03	.725E+04	-.312E+01	.484E+01	.209E+08
234.00	-.134E-01	.222E+03	.665E+04	-.295E+01	.133E+02	.209E+08
252.00	-.185E-01	.172E+03	.609E+04	-.267E+01	.183E+02	.209E+08
270.00	-.209E-01	.127E+03	.560E+04	-.231E+01	.207E+02	.209E+08
288.00	-.214E-01	.890E+02	.517E+04	-.193E+01	.212E+02	.209E+08
306.00	-.204E-01	.576E+02	.482E+04	-.156E+01	.202E+02	.209E+08
324.00	-.186E-01	.325E+02	.455E+04	-.121E+01	.184E+02	.209E+08
342.00	-.162E-01	.134E+02	.433E+04	-.898E+00	.161E+02	.209E+08
360.00	-.137E-01	-.516E+00	.419E+04	-.632E+00	.135E+02	.209E+08
378.00	-.111E-01	-.101E+02	.430E+04	-.411E+00	.110E+02	.209E+08
396.00	-.866E-02	-.161E+02	.436E+04	-.236E+00	.858E+01	.209E+08
414.00	-.649E-02	-.192E+02	.440E+04	-.101E+00	.642E+01	.209E+08
432.00	-.461E-02	-.203E+02	.441E+04	-.256E-02	.457E+01	.209E+08
450.00	-.305E-02	-.198E+02	.440E+04	.653E-01	.302E+01	.209E+08
468.00	-.179E-02	-.184E+02	.439E+04	.108E+00	.177E+01	.209E+08
486.00	-.820E-03	-.163E+02	.437E+04	.131E+00	.811E+00	.209E+08
504.00	-.993E-04	-.139E+02	.434E+04	.139E+00	.981E-01	.209E+08
522.00	.406E-03	-.115E+02	.431E+04	.136E+00	.402E+00	.209E+08
540.00	.732E-03	-.913E+01	.429E+04	.126E+00	.725E+00	.209E+08
558.00	.917E-03	-.702E+01	.426E+04	.111E+00	.908E+00	.209E+08
576.00	.993E-03	-.519E+01	.424E+04	.935E-01	.983E+00	.209E+08
594.00	.987E-03	-.366E+01	.422E+04	.758E-01	.977E+00	.209E+08
612.00	.925E-03	-.245E+01	.421E+04	.587E-01	.916E+00	.209E+08
630.00	.824E-03	-.153E+01	.420E+04	.431E-01	.816E+00	.209E+08
648.00	.699E-03	-.866E+00	.419E+04	.295E-01	.692E+00	.209E+08
666.00	.561E-03	-.427E+00	.419E+04	.182E-01	.555E+00	.209E+08
684.00	.415E-03	-.168E+00	.419E+04	.952E-02	.411E+00	.209E+08
702.00	.267E-03	-.410E-01	.418E+04	.344E-02	.265E+00	.209E+08
720.00	.118E-03	.000E+00	.418E+04	.000E+00	.117E+00	.209E+08

COMPUTED LATERAL FORCE AT PILE HEAD = .50000E+01 KIP

COMPUTED MOMENT AT PILE HEAD = .00000E+00 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.31736E-02

THE OVERALL MOMENT IMBALANCE = -.162E-08 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .117E-07 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .453E+00 IN
MAXIMUM BENDING MOMENT = .475E+03 IN-KIP
MAXIMUM TOTAL STRESS = .826E+04 LBS/IN**2

NO. OF ITERATIONS = 6
MAXIMUM DEFLECTION ERROR = .881E-04 IN

S U M M A R Y   T A B L E
****

LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.500E+01	.000E+00	.100E+03	.453E+00	-.317E-02	.475E+03	.826E+04

EXAMPLE 5, COMPUTE ULTIMATE BENDING MOMENT FOR BORED PILES

Example 5 is included to illustrate the functions of Program COM624P for computing the ultimate bending moment and an interaction diagram. A total of eight axial loads are specified for the program to compute the ultimate bending moment at each axial load and to construct the interaction diagram (ultimate bending moment versus axial load). Only the tables of output for axial load of 0 kips, 100 kips, and 500 kips are shown in the following pages.

The ultimate bending moment of a reinforced-concrete section is taken at a maximum strain of concrete of 0.003 based on the ACI code. It should be noted that the flexural rigidity (EI), corresponding to the ultimate bending moment, is significantly lower than that of the uncracked EI value. Therefore, the user should also pay attention to the variation of EI versus moment as shown in the first two columns in the output summary. In general, the moment distribution is not much affected by the EI used in the computation. However, if the deflection is more critical for the design, then careful interpretation of EI should be done.

Three ranges of EI magnitude can be found in the output. The first range of EI magnitude is associated with the uncracked stage. The concrete is uncracked and the EI is more-or-less constant and is equal to the calculated EI for the gross section. The second range of EI magnitude is for the cracked stage. A significant decrease in the EI value takes place as cracks continue propagating. The third range of EI magnitude is for the cracked and large strain stage. The EI value is further reduced because the stress-strain curve as shown in Fig. 4.1 of Part II of this manual is softened at large strain.

The input and output data are shown in the following pages.

EXAMPLE 5 COMPUTE ULTIMATE BENDING MOMENT FOR BORED PILES

****
ULTIMATE BENDING RESISTANCE AND FLEXURAL RIGIDITY
****

DIAMETER = 30.00 IN

CONCRETE COMPRESSIVE STRENGTH = 4.000000 KIP/IN**2

REBAR YIELD STRENGTH = 60.000000 KIP/IN**2

MODULUS OF ELASTICITY OF STEEL = 29000.000000 KIP/IN**2

NUMBER OF REINFORCING BARS = 12

NUMBER OF ROWS OF REINFORCING BARS = 7

COVER THICKNESS = 3.000 IN
SQUASH LOAD CAPACITY = 2939.89 KIPS

ROW NUMBER	AREA OF REINFORCEMENT IN**2	DISTANCE TO CENTROIDAL AXIS IN
1	.790000	12.0000
2	1.580000	10.3923
3	1.580000	6.0000
4	1.580000	.0000
5	1.580000	-6.0000
6	1.580000	-10.3923
7	.790000	-12.0000

OUTPUT RESULTS FOR AN AXIAL LOAD = .00 KIPS
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
164.369	.16437E+09	.000001	.00002	15.0477
813.938	.16279E+09	.000005	.00008	15.0473
1450.856	.16121E+09	.000009	.00014	15.0470
1450.856	.11160E+09	.000013	.00011	8.2641
1450.856	.85344E+08	.000017	.00014	8.2758
1450.856	.69088E+08	.000021	.00017	8.2877
1450.856	.58034E+08	.000025	.00021	8.2998
1450.856	.50030E+08	.000029	.00024	8.3120
1450.856	.43965E+08	.000033	.00027	8.3241
1542.059	.41677E+08	.000037	.00031	8.3367

1706.145	.41613E+08	.000041	.00034	8.3490
1869.770	.41550E+08	.000045	.00038	8.3619
2032.827	.41486E+08	.000049	.00041	8.3747
2195.364	.41422E+08	.000053	.00044	8.3877
3396.051	.40916E+08	.000083	.00070	8.4898
4557.698	.40334E+08	.000113	.00097	8.5752
5181.602	.36235E+08	.000143	.00120	8.4079
5551.161	.32088E+08	.000173	.00141	8.1675
5731.058	.28232E+08	.000203	.00161	7.9192
5896.307	.25306E+08	.000233	.00180	7.7198
6051.316	.23009E+08	.000263	.00199	7.5807
6143.448	.20967E+08	.000293	.00218	7.4550
6173.590	.19113E+08	.000323	.00235	7.2786
6201.067	.17567E+08	.000353	.00252	7.1394
6226.547	.16257E+08	.000383	.00269	7.0284
6253.610	.15142E+08	.000413	.00288	6.9714
6274.235	.14163E+08	.000443	.00305	6.8853
6293.935	.13306E+08	.000473	.00322	6.8144
6312.940	.12551E+08	.000503	.00340	6.7549
6331.056	.11878E+08	.000533	.00357	6.7038
6348.535	.11276E+08	.000563	.00375	6.6613
6359.241	.10724E+08	.000593	.00394	6.6402

THE ULTIMATE BENDING MOMENT AT A CONCRETE STRAIN OF 0.003
IS :   .627E+04 IN-KIP

OUTPUT RESULTS FOR AN AXIAL LOAD =   100.00 KIPS
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
162.006	.16201E+09	.000001	.00005	50.6153
808.512	.16170E+09	.000005	.00011	22.2220
1444.742	.16053E+09	.000009	.00017	19.0790
1444.742	.11113E+09	.000013	.00019	14.9310
1444.742	.84985E+08	.000017	.00023	13.6948
1520.720	.72415E+08	.000021	.00027	12.8775
1691.928	.67677E+08	.000025	.00031	12.2856
1860.528	.64156E+08	.000029	.00034	11.8438
2028.040	.61456E+08	.000033	.00038	11.5001
2191.405	.59227E+08	.000037	.00042	11.2183
2355.083	.57441E+08	.000041	.00045	10.9999
2517.698	.55949E+08	.000045	.00049	10.8075
2678.916	.54672E+08	.000049	.00052	10.6459
2839.518	.53576E+08	.000053	.00056	10.5106
4022.045	.48458E+08	.000083	.00083	9.9455
5163.315	.45693E+08	.000113	.00110	9.7123

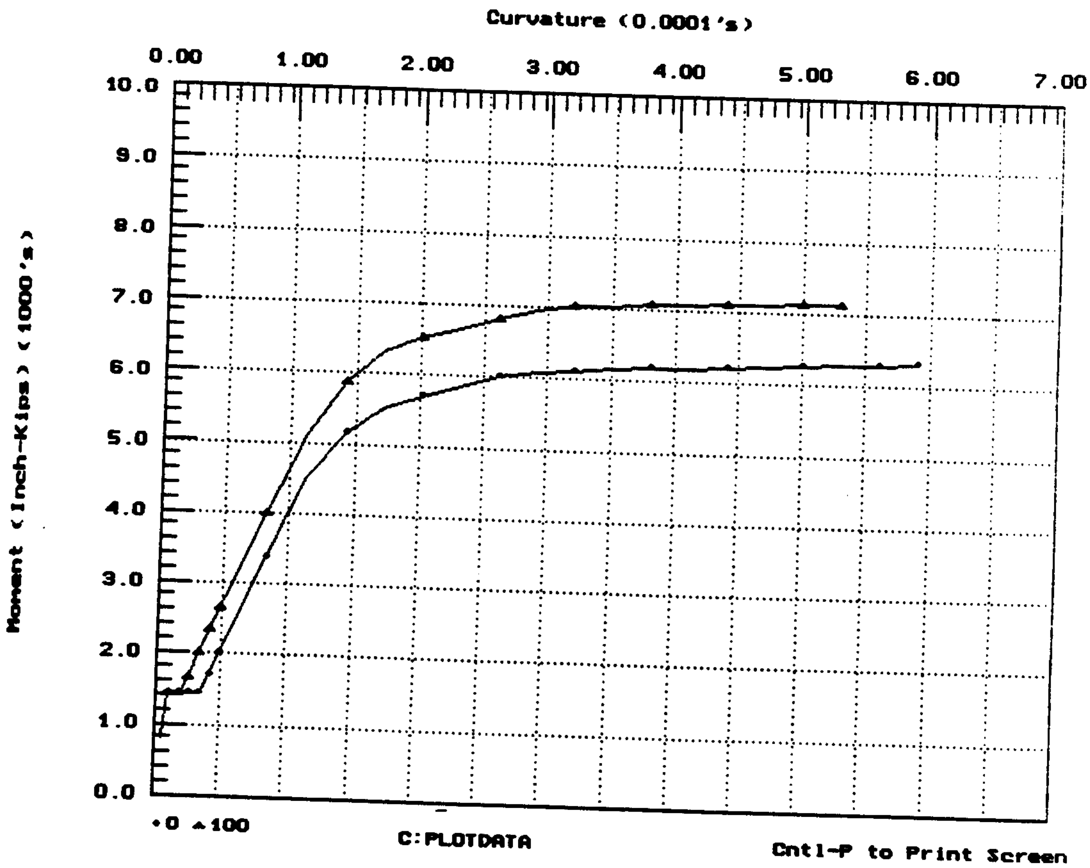
5932.108	.41483E+08	.000143	.00135	9.4648
6379.425	.36875E+08	.000173	.00159	9.1707
6569.130	.32360E+08	.000203	.00180	8.8503
6723.582	.28857E+08	.000233	.00200	8.5933
6868.083	.26114E+08	.000263	.00222	8.4326
6997.694	.23883E+08	.000293	.00243	8.2930
7067.918	.21882E+08	.000323	.00263	8.1401
7081.734	.20062E+08	.000353	.00282	8.0000
7120.547	.18592E+08	.000383	.00302	7.8743
7140.321	.17289E+08	.000413	.00320	7.7537
7158.454	.16159E+08	.000443	.00339	7.6537
7168.455	.15155E+08	.000473	.00359	7.5871
7176.483	.14267E+08	.000503	.00379	7.5333
7176.483	.13464E+08	.000533	.00400	7.4999

THE ULTIMATE BENDING MOMENT AT A CONCRETE STRAIN OF 0.003
IS :   .712E+04 IN-KIP

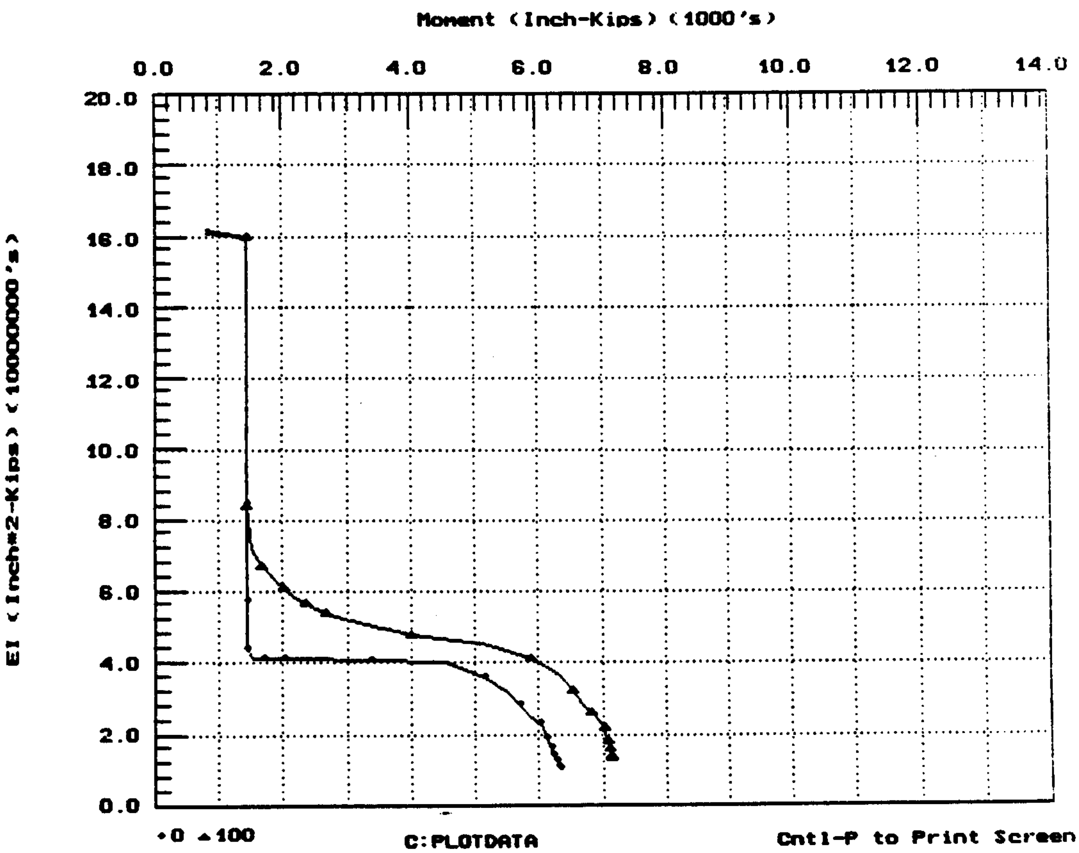
OUTPUT RESULTS FOR AN AXIAL LOAD =   500.00 KIPS
****

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
150.462	.15046E+09	.000001	.00020	*******
752.095	.15042E+09	.000005	.00026	52.0128
1353.050	.15034E+09	.000009	.00032	35.6773
1953.446	.15027E+09	.000013	.00038	29.4320
2549.110	.14995E+09	.000017	.00044	26.1480
3137.984	.14943E+09	.000021	.00051	24.1300
3243.175	.12973E+09	.000025	.00055	22.0952
3542.862	.12217E+09	.000029	.00060	20.8238
3808.285	.11540E+09	.000033	.00065	19.8069
4051.712	.10951E+09	.000037	.00070	18.9774
4273.498	.10423E+09	.000041	.00075	18.2747
4482.946	.99621E+08	.000045	.00080	17.6794
4682.184	.95555E+08	.000049	.00084	17.1674
4873.092	.91945E+08	.000053	.00089	16.7220
6133.245	.73895E+08	.000083	.00121	14.6076
7229.911	.63982E+08	.000113	.00153	13.5570
8225.903	.57524E+08	.000143	.00186	12.9841
8857.891	.51202E+08	.000173	.00217	12.5170
9201.317	.45327E+08	.000203	.00246	12.1343
9481.273	.40692E+08	.000233	.00276	11.8621
9562.766	.36360E+08	.000263	.00305	11.5895
9637.997	.32894E+08	.000293	.00334	11.4096
9678.995	.29966E+08	.000323	.00364	11.2737
9692.204	.27457E+08	.000353	.00396	11.2071

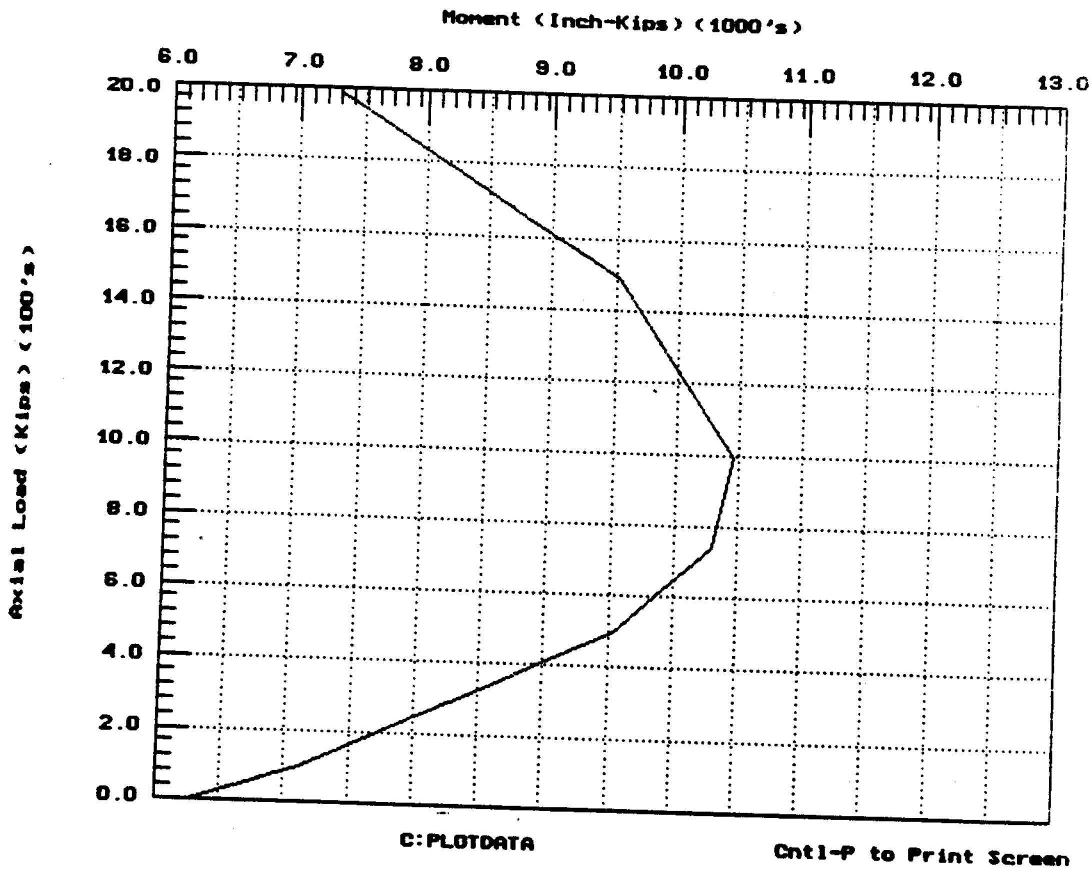
THE ULTIMATE BENDING MOMENT AT A CONCRETE STRAIN OF 0.003
IS :   .955E+04 IN-KIP



Bending Moment vs. Curvature



EI vs. Bending Moment



Interaction Diagram

EXAMPLE 6, ANALYSES RELATED TO DESIGN OF CONCRETE PILES

This example is presented to illustrate the capability of Program COM624P to perform analyses that can yield results of direct benefit to the designer of a reinforced-concrete pile. The pile is 30-inches in diameter and 30 ft in length. The pile is embedded in a uniform dense sand with an internal friction angle of 38 degrees. In general, with input information provided for reinforcement in the same data file, the program will compute the ultimate bending moment as the first step. Loadings and preliminary data on piles are selected, and the program yields values of pile deflection, moment, shear, and soil resistance as the second step.

The user can then compare the maximum bending moment computed in the second step with the ultimate bending moment in the first step for an allowable factor of safety. The properties of the pile can then be changed, if necessary or desirable, and further computations made to achieve the final selection of the properties of the pile.

As described in Example 5, the EI values used for each pile have a significant effect on the deflection of the pile. The relationship between moment and EI is computed during the first step. Therefore, the user can ask the program to take the moment-EI variation directly into the computation. The user may also manually input the justified EI values for different sections, based on the curve of bending moment versus depth obtained earlier. In this example, the option for automatic iterations using internally-generated, cracked/uncracked EI values was given to the computer for the final solution.

EXAMPLE 6 ANALYSES RELATED TO DESIGN OF CONCRETE PILES

```
*****************************************************************
ULTIMATE BENDING RESISTANCE AND FLEXURAL RIGIDITY
*****************************************************************
```

DIAMETER = 30.00 IN

CONCRETE COMPRESSIVE STRENGTH = 3.500000 KIP/IN**2

REBAR YIELD STRENGTH = 60.000000 KIP/IN**2

MODULUS OF ELASTICITY OF STEEL = 29000.000000 KIP/IN**2

NUMBER OF REINFORCING BARS = 12

NUMBER OF ROWS OF REINFORCING BARS = 7

COVER THICKNESS = 3.000 IN
SQUASH LOAD CAPACITY = 2643.50 KIP

ROW NUMBER	AREA OF REINFORCEMENT IN**2	DISTANCE TO CENTROIDAL AXIS IN
1	.790000	12.0000
2	1.580000	10.3923
3	1.580000	6.0000
4	1.580000	.0000
5	1.580000	-6.0000
6	1.580000	-10.3923
7	.790000	-12.0000

OUTPUT RESULTS FOR AN AXIAL LOAD = 50.00 KIP
```
******************************************************
```

MOMENT IN-KIP	EI KIP-IN**2	PHI 1/IN	MAX STR IN/IN	N AXIS IN
153.941	.15394E+09	.000001	.00003	33.870
765.633	.15313E+09	.000005	.00009	18.860
1364.857	.15165E+09	.000009	.00015	17.192
1364.857	.10499E+09	.000013	.00016	12.334
1364.857	.80286E+08	.000017	.00020	11.553
1364.857	.64993E+08	.000021	.00023	11.048
1364.857	.54594E+08	.000025	.00027	10.696
1517.223	.52318E+08	.000029	.00030	10.444

1678.489	.50863E+08	.000033	.00034	10.238
1839.116	.49706E+08	.000037	.00037	10.079
1999.618	.48771E+08	.000041	.00041	9.965
2158.825	.47974E+08	.000045	.00044	9.859
2317.381	.47293E+08	.000049	.00048	9.772
2475.348	.46705E+08	.000053	.00051	9.701
3639.372	.43848E+08	.000083	.00078	9.441
4762.959	.42150E+08	.000113	.00106	9.375
5467.619	.38235E+08	.000143	.00131	9.170
5896.697	.34085E+08	.000173	.00155	8.955
6057.286	.29839E+08	.000203	.00175	8.636
6206.912	.26639E+08	.000233	.00197	8.437
6340.834	.24110E+08	.000263	.00218	8.274
6466.427	.22070E+08	.000293	.00239	8.161
6519.455	.20184E+08	.000323	.00259	8.011
6547.419	.18548E+08	.000353	.00278	7.887
6568.075	.17149E+08	.000383	.00297	7.753
6587.674	.15951E+08	.000413	.00316	7.644
6606.533	.14913E+08	.000443	.00335	7.552
6606.533	.13967E+08	.000473	.00355	7.500
6631.914	.13185E+08	.000503	.00376	7.480
6639.605	.12457E+08	.000533	.00396	7.434

THE ULTIMATE BENDING MOMENT AT A CONCRETE STRAIN OF 0.003
IS :   .657E+04 IN-KIP

EXAMPLE 6 ANALYSES RELATED TO DESIGN OF CONCRETE PILES

UNITS--ENGL

****
PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE
****

I N P U T   I N F O R M A T I O N
****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 360.00 IN
MODULUS OF ELASTICITY OF PILE = .320E+04 KIP/IN**2
1 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00			
	30.000	.398E+05	.707E+03
360.00			

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = .00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

2 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A SAND
X AT THE TOP OF THE LAYER = .00 IN
X AT THE BOTTOM OF THE LAYER = 360.00 IN
VARIATION OF SOIL MODULUS, k = .900E+02 LBS/IN**3

LAYER 2
THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
X AT THE TOP OF THE LAYER = 360.00 IN
X AT THE BOTTOM OF THE LAYER = 540.00 IN
VARIATION OF SOIL MODULUS, k = .500E+03 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
4 POINTS

X,IN	WEIGHT,LBS/IN**3
.00	.69E-01
360.00	.69E-01
360.00	.34E-01
540.00	.34E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
4 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
.00	.000E+00	38.000	-----
360.00	.000E+00	38.000	-----
360.00	.100E+02	.000	.500E-02
540.00	.100E+02	.000	.500E-02

FINITE DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS	=	100
TOLERANCE ON DETERMINATION OF DEFLECTIONS	=	.100E-04 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS	=	100
MAXIMUM ALLOWABLE DEFLECTION	=	.15E+03 IN

INPUT CODES

OUTPT = 1
KCYCL = 1
KBC = 1
KPYOP = 1
INC = 1

EXAMPLE 6 ANALYSES RELATED TO DESIGN OF CONCRETE PILES

UNITS--ENGL

O U T P U T   I N F O R M A T I O N
****

GENERATED P-Y CURVES

THE NUMBER OF CURVE IS = 4

THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B
20.00	30.00	38.0	.7E-01	2.36	1.73

Y IN	P LBS/IN
.000	.000
.042	75.000
.083	150.000
.125	225.000
.167	300.000
.208	365.158
.250	384.822
.292	402.273
.333	418.027
.375	432.435
.417	445.743
.458	458.134
.500	469.747
1.125	638.676
31.125	638.676
61.125	638.676
91.125	638.676

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B
60.00	30.00	38.0	.7E-01	1.48	1.05

Y IN	P LBS/IN
.000	.000
.042	225.000
.083	450.000
.125	675.000
.167	900.000
.208	1125.000
.250	1216.512
.292	1279.527
.333	1336.746
.375	1389.336
.417	1438.131
.458	1483.745
.500	1526.650
1.125	2151.850
31.125	2151.850
61.125	2151.850
91.125	2151.850

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B
100.00	30.00	38.0	.7E-01	1.01	.65

Y IN	P LBS/IN
.000	.000
.042	375.000
.083	750.000
.125	1125.000
.167	1401.127
.208	1545.320
.250	1674.083
.292	1791.286
.333	1899.424
.375	2000.215
.417	2094.899
.458	2184.406
.500	2269.455
1.125	3514.745
31.125	3514.745
61.125	3514.745
91.125	3514.745

DEPTH BELOW GS IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B

150.00	30.00	38.0	.7E-01	.88	.50

Y IN	P LBS/IN
.000	.000
.042	562.500
.083	1125.000
.125	1558.341
.167	1856.202
.208	2125.918
.250	2375.137
.292	2608.509
.333	2829.120
.375	3039.150
.417	3240.206
.458	3433.519
.500	3620.052
1.125	6371.292
31.125	6371.292
61.125	6371.292
91.125	6371.292

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .200E+02 KIP
APPLIED MOMENT AT PILE HEAD = .000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD = .500E+02 KIP

X IN *****	DEFLECTION IN **********	MOMENT IN-KIP **********	TOTAL STRESS LBS/IN**2 **********	SHEAR KIP **********	SOIL RESIST LBS/IN **********	FLEXURAL RIGIDITY KIP-IN**2 **********
.00	.110E+00	.000E+00	.707E+02	.201E+02	.000E+00	.153E+09
3.60	.106E+00	.000E+00	.707E+02	.200E+02	.345E+02	.153E+09
7.20	.103E+00	.144E+03	.125E+03	.198E+02	.665E+02	.153E+09
10.80	.989E-01	.215E+03	.152E+03	.195E+02	.961E+02	.153E+09
14.40	.951E-01	.284E+03	.178E+03	.191E+02	.123E+03	.153E+09
18.00	.914E-01	.352E+03	.204E+03	.186E+02	.148E+03	.153E+09
21.60	.877E-01	.419E+03	.229E+03	.180E+02	.171E+03	.153E+09
25.20	.841E-01	.482E+03	.253E+03	.174E+02	.191E+03	.153E+09

28.80	.805E-01	.544E+03	.276E+03	.166E+02	.209E+03	.153E+09
32.40	.769E-01	.603E+03	.298E+03	.159E+02	.224E+03	.153E+09
36.00	.734E-01	.658E+03	.319E+03	.150E+02	.238E+03	.153E+09
39.60	.700E-01	.711E+03	.339E+03	.141E+02	.249E+03	.153E+09
43.20	.666E-01	.761E+03	.358E+03	.132E+02	.259E+03	.153E+09
46.80	.633E-01	.807E+03	.375E+03	.123E+02	.266E+03	.153E+09
50.40	.600E-01	.849E+03	.391E+03	.113E+02	.272E+03	.153E+09
54.00	.568E-01	.889E+03	.406E+03	.103E+02	.276E+03	.153E+09
57.60	.537E-01	.924E+03	.419E+03	.933E+01	.278E+03	.153E+09
61.20	.507E-01	.956E+03	.431E+03	.833E+01	.279E+03	.153E+09
64.80	.477E-01	.984E+03	.442E+03	.733E+01	.278E+03	.153E+09
68.40	.448E-01	.101E+04	.451E+03	.633E+01	.276E+03	.153E+09
72.00	.421E-01	.103E+04	.459E+03	.534E+01	.273E+03	.152E+09
75.60	.394E-01	.105E+04	.466E+03	.437E+01	.268E+03	.152E+09
79.20	.368E-01	.106E+04	.471E+03	.342E+01	.262E+03	.152E+09
82.80	.343E-01	.107E+04	.475E+03	.248E+01	.255E+03	.152E+09
86.40	.318E-01	.108E+04	.478E+03	.158E+01	.247E+03	.152E+09
90.00	.295E-01	.108E+04	.480E+03	.704E+00	.239E+03	.152E+09
93.60	.273E-01	.109E+04	.480E+03	-.139E+00	.230E+03	.152E+09
97.20	.251E-01	.108E+04	.480E+03	-.948E+00	.220E+03	.152E+09
100.80	.231E-01	.108E+04	.478E+03	-.172E+01	.209E+03	.152E+09
104.40	.211E-01	.107E+04	.475E+03	-.245E+01	.198E+03	.152E+09
108.00	.192E-01	.106E+04	.471E+03	-.315E+01	.187E+03	.152E+09
111.60	.174E-01	.105E+04	.466E+03	-.380E+01	.175E+03	.152E+09
115.20	.157E-01	.103E+04	.461E+03	-.441E+01	.163E+03	.152E+09
118.80	.141E-01	.102E+04	.455E+03	-.497E+01	.151E+03	.153E+09
122.40	.126E-01	.999E+03	.447E+03	-.550E+01	.139E+03	.153E+09
126.00	.112E-01	.978E+03	.440E+03	-.597E+01	.127E+03	.153E+09
129.60	.984E-02	.956E+03	.431E+03	-.641E+01	.115E+03	.153E+09
133.20	.857E-02	.932E+03	.422E+03	-.680E+01	.103E+03	.153E+09
136.80	.738E-02	.907E+03	.413E+03	-.715E+01	.909E+02	.153E+09
140.40	.627E-02	.881E+03	.403E+03	-.745E+01	.792E+02	.153E+09
144.00	.523E-02	.853E+03	.393E+03	-.772E+01	.678E+02	.153E+09
147.60	.426E-02	.825E+03	.382E+03	-.794E+01	.566E+02	.153E+09
151.20	.337E-02	.796E+03	.371E+03	-.813E+01	.458E+02	.153E+09
154.80	.254E-02	.767E+03	.360E+03	-.827E+01	.354E+02	.153E+09
158.40	.177E-02	.737E+03	.349E+03	-.838E+01	.253E+02	.153E+09
162.00	.107E-02	.706E+03	.337E+03	-.846E+01	.156E+02	.153E+09
165.60	.430E-03	.676E+03	.326E+03	-.850E+01	.641E+01	.153E+09
169.20	-.154E-03	.645E+03	.314E+03	-.850E+01	-.235E+01	.153E+09
172.80	-.684E-03	.615E+03	.303E+03	-.848E+01	-.106E+02	.153E+09
176.40	-.116E-02	.584E+03	.291E+03	-.843E+01	-.184E+02	.153E+09
180.00	-.159E-02	.554E+03	.280E+03	-.835E+01	-.258E+02	.153E+09
183.60	-.197E-02	.524E+03	.268E+03	-.824E+01	-.326E+02	.153E+09
187.20	-.231E-02	.495E+03	.257E+03	-.811E+01	-.389E+02	.153E+09
190.80	-.260E-02	.466E+03	.246E+03	-.796E+01	-.447E+02	.153E+09
194.40	-.286E-02	.437E+03	.236E+03	-.779E+01	-.500E+02	.153E+09
198.00	-.308E-02	.410E+03	.225E+03	-.760E+01	-.549E+02	.153E+09
201.60	-.326E-02	.383E+03	.215E+03	-.740E+01	-.592E+02	.153E+09
205.20	-.342E-02	.357E+03	.205E+03	-.718E+01	-.631E+02	.153E+09
208.80	-.354E-02	.331E+03	.196E+03	-.695E+01	-.665E+02	.153E+09
212.40	-.363E-02	.307E+03	.186E+03	-.670E+01	-.694E+02	.153E+09

216.00	-.370E-02	.283E+03	.177E+03	-.645E+01	-.719E+02	.153E+09
219.60	-.374E-02	.260E+03	.169E+03	-.618E+01	-.740E+02	.153E+09
223.20	-.376E-02	.238E+03	.161E+03	-.592E+01	-.756E+02	.153E+09
226.80	-.376E-02	.218E+03	.153E+03	-.564E+01	-.769E+02	.153E+09
230.40	-.375E-02	.198E+03	.145E+03	-.536E+01	-.777E+02	.153E+09
234.00	-.371E-02	.179E+03	.138E+03	-.508E+01	-.782E+02	.153E+09
237.60	-.367E-02	.161E+03	.131E+03	-.480E+01	-.784E+02	.153E+09
241.20	-.360E-02	.144E+03	.125E+03	-.452E+01	-.782E+02	.153E+09
244.80	-.353E-02	.129E+03	.119E+03	-.424E+01	-.777E+02	.153E+09
248.40	-.344E-02	.114E+03	.114E+03	-.396E+01	-.770E+02	.153E+09
252.00	-.335E-02	.100E+03	.108E+03	-.368E+01	-.759E+02	.153E+09
255.60	-.324E-02	.873E+02	.104E+03	-.341E+01	-.746E+02	.153E+09
259.20	-.313E-02	.755E+02	.992E+02	-.315E+01	-.731E+02	.153E+09
262.80	-.302E-02	.646E+02	.951E+02	-.289E+01	-.713E+02	.153E+09
266.40	-.289E-02	.547E+02	.913E+02	-.263E+01	-.694E+02	.153E+09
270.00	-.277E-02	.456E+02	.879E+02	-.239E+01	-.672E+02	.153E+09
273.60	-.263E-02	.375E+02	.849E+02	-.215E+01	-.649E+02	.153E+09
277.20	-.250E-02	.301E+02	.821E+02	-.192E+01	-.624E+02	.153E+09
280.80	-.236E-02	.236E+02	.796E+02	-.170E+01	-.597E+02	.153E+09
284.40	-.222E-02	.179E+02	.775E+02	-.149E+01	-.569E+02	.153E+09
288.00	-.208E-02	.129E+02	.756E+02	-.129E+01	-.540E+02	.153E+09
291.60	-.194E-02	.857E+01	.740E+02	-.110E+01	-.510E+02	.153E+09
295.20	-.180E-02	.492E+01	.726E+02	-.925E+00	-.478E+02	.153E+09
298.80	-.166E-02	.190E+01	.714E+02	-.758E+00	-.445E+02	.153E+09
302.40	-.151E-02	-.552E+00	.709E+02	-.604E+00	-.412E+02	.153E+09
306.00	-.137E-02	-.247E+01	.717E+02	-.462E+00	-.378E+02	.153E+09
309.60	-.123E-02	-.389E+01	.722E+02	-.332E+00	-.342E+02	.153E+09
313.20	-.109E-02	-.487E+01	.726E+02	-.216E+00	-.306E+02	.153E+09
316.80	-.944E-03	-.546E+01	.728E+02	-.112E+00	-.269E+02	.153E+09
320.40	-.802E-03	-.570E+01	.729E+02	-.222E-01	-.231E+02	.153E+09
324.00	-.661E-03	-.563E+01	.728E+02	.542E-01	-.193E+02	.153E+09
327.60	-.521E-03	-.532E+01	.727E+02	.117E+00	-.154E+02	.153E+09
331.20	-.381E-03	-.481E+01	.725E+02	.165E+00	-.113E+02	.153E+09
334.80	-.241E-03	-.415E+01	.723E+02	.198E+00	-.726E+01	.153E+09
338.40	-.102E-03	-.340E+01	.720E+02	.217E+00	-.310E+01	.153E+09
342.00	.374E-04	-.260E+01	.717E+02	.220E+00	.115E+01	.153E+09
345.60	.176E-03	-.183E+01	.714E+02	.208E+00	.548E+01	.153E+09
349.20	.315E-03	-.112E+01	.711E+02	.181E+00	.990E+01	.153E+09
352.80	.453E-03	-.539E+00	.709E+02	.137E+00	.144E+02	.153E+09
356.40	.592E-03	.000E+00	.707E+02	.000E+00	.190E+02	.153E+09
360.00	.730E-03	.000E+00	.707E+02	.000E+00	.237E+02	.153E+09

COMPUTED LATERAL FORCE AT PILE HEAD = .20000E+02 KIP
COMPUTED MOMENT AT PILE HEAD = .00000E+00 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.10473E-02

THE OVERALL MOMENT IMBALANCE = -.813E-08 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .129E-06 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .110E+00 IN
MAXIMUM BENDING MOMENT = .112E+04 IN-KIP
MAXIMUM TOTAL STRESS = .493E+03 LBS/IN**2

NO. OF ITERATIONS = 6
MAXIMUM DEFLECTION ERROR = .461E-07 IN

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .400E+02 KIP
APPLIED MOMENT AT PILE HEAD = .000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD = .500E+02 KIP

X IN	DEFLECTION IN	MOMENT IN-KIP	TOTAL STRESS LBS/IN**2	SHEAR KIP	SOIL RESIST LBS/IN	FLEXURAL RIGIDITY KIP-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	.337E+00	.000E+00	.707E+02	.402E+02	.000E+00	.153E+09
3.60	.323E+00	.000E+00	.707E+02	.401E+02	.608E+02	.153E+09
7.20	.309E+00	.289E+03	.180E+03	.396E+02	.128E+03	.153E+09
10.80	.296E+00	.431E+03	.233E+03	.390E+02	.199E+03	.153E+09
14.40	.282E+00	.570E+03	.286E+03	.381E+02	.272E+03	.153E+09
18.00	.269E+00	.707E+03	.337E+03	.370E+02	.348E+03	.153E+09
21.60	.255E+00	.838E+03	.387E+03	.356E+02	.423E+03	.153E+09
25.20	.242E+00	.964E+03	.435E+03	.340E+02	.497E+03	.153E+09
28.80	.228E+00	.108E+04	.480E+03	.320E+02	.568E+03	.153E+09
32.40	.215E+00	.120E+04	.522E+03	.299E+02	.628E+03	.152E+09
36.00	.202E+00	.130E+04	.561E+03	.276E+02	.655E+03	.152E+09
39.60	.189E+00	.140E+04	.597E+03	.252E+02	.674E+03	.152E+09
43.20	.176E+00	.148E+04	.630E+03	.227E+02	.686E+03	.541E+08
46.80	.164E+00	.156E+04	.660E+03	.203E+02	.691E+03	.528E+08
50.40	.152E+00	.163E+04	.686E+03	.178E+02	.690E+03	.519E+08
54.00	.141E+00	.169E+04	.708E+03	.153E+02	.684E+03	.513E+08
57.60	.130E+00	.174E+04	.728E+03	.129E+02	.672E+03	.508E+08
61.20	.119E+00	.178E+04	.744E+03	.105E+02	.655E+03	.504E+08
64.80	.109E+00	.182E+04	.757E+03	.815E+01	.634E+03	.501E+08
68.40	.991E-01	.184E+04	.766E+03	.591E+01	.610E+03	.498E+08
72.00	.899E-01	.186E+04	.773E+03	.377E+01	.582E+03	.496E+08

75.60	.811E-01	.187E+04	.777E+03	.172E+01	.552E+03	.495E+08
79.20	.729E-01	.187E+04	.778E+03	-.205E+00	.520E+03	.494E+08
82.80	.651E-01	.187E+04	.777E+03	-.201E+01	.485E+03	.495E+08
86.40	.579E-01	.186E+04	.773E+03	-.370E+01	.450E+03	.495E+08
90.00	.511E-01	.185E+04	.767E+03	-.525E+01	.414E+03	.496E+08
93.60	.448E-01	.182E+04	.759E+03	-.668E+01	.378E+03	.497E+08
97.20	.390E-01	.180E+04	.749E+03	-.797E+01	.341E+03	.499E+08
100.80	.337E-01	.177E+04	.737E+03	-.914E+01	.305E+03	.501E+08
104.40	.288E-01	.173E+04	.724E+03	-.102E+02	.270E+03	.503E+08
108.00	.243E-01	.169E+04	.710E+03	-.111E+02	.236E+03	.506E+08
111.60	.203E-01	.165E+04	.694E+03	-.119E+02	.204E+03	.509E+08
115.20	.167E-01	.161E+04	.678E+03	-.126E+02	.173E+03	.513E+08
118.80	.135E-01	.156E+04	.660E+03	-.131E+02	.145E+03	.517E+08
122.40	.108E-01	.151E+04	.642E+03	-.136E+02	.119E+03	.521E+08
126.00	.835E-02	.147E+04	.624E+03	-.140E+02	.946E+02	.527E+08
129.60	.629E-02	.141E+04	.604E+03	-.143E+02	.734E+02	.534E+08
133.20	.458E-02	.136E+04	.585E+03	-.145E+02	.550E+02	.542E+08
136.80	.320E-02	.131E+04	.565E+03	-.147E+02	.394E+02	.152E+09
140.40	.193E-02	.126E+04	.545E+03	-.148E+02	.244E+02	.152E+09
144.00	.765E-03	.120E+04	.525E+03	-.149E+02	.991E+01	.152E+09
147.60	-.297E-03	.115E+04	.505E+03	-.149E+02	-.395E+01	.152E+09
151.20	-.126E-02	.110E+04	.484E+03	-.148E+02	-.172E+02	.152E+09
154.80	-.213E-02	.104E+04	.464E+03	-.148E+02	-.297E+02	.152E+09
158.40	-.291E-02	.990E+03	.444E+03	-.146E+02	-.415E+02	.152E+09
162.00	-.361E-02	.938E+03	.425E+03	-.145E+02	-.526E+02	.153E+09
165.60	-.423E-02	.886E+03	.405E+03	-.142E+02	-.630E+02	.153E+09
169.20	-.477E-02	.835E+03	.386E+03	-.140E+02	-.727E+02	.153E+09
172.80	-.524E-02	.786E+03	.367E+03	-.137E+02	-.815E+02	.153E+09
176.40	-.565E-02	.737E+03	.349E+03	-.134E+02	-.897E+02	.153E+09
180.00	-.599E-02	.689E+03	.331E+03	-.131E+02	-.971E+02	.153E+09
183.60	-.628E-02	.643E+03	.313E+03	-.127E+02	-.104E+03	.153E+09
187.20	-.651E-02	.597E+03	.296E+03	-.123E+02	-.110E+03	.153E+09
190.80	-.669E-02	.554E+03	.280E+03	-.119E+02	-.115E+03	.153E+09
194.40	-.682E-02	.512E+03	.264E+03	-.115E+02	-.119E+03	.153E+09
198.00	-.691E-02	.471E+03	.248E+03	-.111E+02	-.123E+03	.153E+09
201.60	-.696E-02	.432E+03	.234E+03	-.106E+02	-.126E+03	.153E+09
205.20	-.697E-02	.394E+03	.219E+03	-.102E+02	-.129E+03	.153E+09
208.80	-.695E-02	.359E+03	.206E+03	-.970E+01	-.131E+03	.153E+09
212.40	-.690E-02	.324E+03	.193E+03	-.923E+01	-.132E+03	.153E+09
216.00	-.682E-02	.292E+03	.181E+03	-.875E+01	-.133E+03	.153E+09
219.60	-.672E-02	.261E+03	.169E+03	-.827E+01	-.133E+03	.153E+09
223.20	-.660E-02	.233E+03	.158E+03	-.780E+01	-.132E+03	.153E+09
226.80	-.645E-02	.205E+03	.148E+03	-.732E+01	-.132E+03	.153E+09
230.40	-.629E-02	.180E+03	.139E+03	-.685E+01	-.130E+03	.153E+09
234.00	-.611E-02	.156E+03	.130E+03	-.638E+01	-.129E+03	.153E+09
237.60	-.592E-02	.134E+03	.121E+03	-.592E+01	-.127E+03	.153E+09
241.20	-.572E-02	.113E+03	.113E+03	-.547E+01	-.124E+03	.153E+09
244.80	-.551E-02	.944E+02	.106E+03	-.503E+01	-.121E+03	.153E+09
248.40	-.529E-02	.770E+02	.998E+02	-.460E+01	-.118E+03	.153E+09
252.00	-.506E-02	.612E+02	.938E+02	-.418E+01	-.115E+03	.153E+09
255.60	-.483E-02	.469E+02	.884E+02	-.377E+01	-.111E+03	.153E+09
259.20	-.460E-02	.340E+02	.836E+02	-.338E+01	-.107E+03	.153E+09

262.80	-.436E-02	.225E+02	.792E+02	-.300E+01	-.103E+03	.153E+09
266.40	-.412E-02	.124E+02	.754E+02	-.264E+01	-.988E+02	.153E+09
270.00	-.388E-02	.353E+01	.721E+02	-.229E+01	-.943E+02	.153E+09
273.60	-.364E-02	-.412E+01	.723E+02	-.196E+01	-.896E+02	.153E+09
277.20	-.340E-02	-.106E+02	.747E+02	-.165E+01	-.848E+02	.153E+09
280.80	-.316E-02	-.160E+02	.768E+02	-.135E+01	-.798E+02	.153E+09
284.40	-.292E-02	-.203E+02	.784E+02	-.107E+01	-.747E+02	.153E+09
288.00	-.268E-02	-.237E+02	.797E+02	-.812E+00	-.695E+02	.153E+09
291.60	-.245E-02	-.262E+02	.806E+02	-.571E+00	-.642E+02	.153E+09
295.20	-.222E-02	-.279E+02	.812E+02	-.349E+00	-.589E+02	.153E+09
298.80	-.199E-02	-.287E+02	.816E+02	-.147E+00	-.534E+02	.153E+09
302.40	-.176E-02	-.289E+02	.816E+02	.348E-01	-.478E+02	.153E+09
306.00	-.153E-02	-.285E+02	.815E+02	.197E+00	-.422E+02	.153E+09
309.60	-.131E-02	-.276E+02	.811E+02	.339E+00	-.365E+02	.153E+09
313.20	-.109E-02	-.261E+02	.806E+02	.459E+00	-.307E+02	.153E+09
316.80	-.871E-03	-.243E+02	.799E+02	.559E+00	-.248E+02	.153E+09
320.40	-.654E-03	-.221E+02	.791E+02	.638E+00	-.189E+02	.153E+09
324.00	-.440E-03	-.197E+02	.782E+02	.695E+00	-.128E+02	.153E+09
327.60	-.227E-03	-.171E+02	.772E+02	.730E+00	-.670E+01	.153E+09
331.20	-.161E-04	-.145E+02	.762E+02	.743E+00	-.480E+00	.153E+09
334.80	.194E-03	-.118E+02	.752E+02	.734E+00	.584E+01	.153E+09
338.40	.403E-03	-.919E+01	.742E+02	.701E+00	.123E+02	.153E+09
342.00	.611E-03	-.676E+01	.733E+02	.645E+00	.188E+02	.153E+09
345.60	.819E-03	-.457E+01	.724E+02	.565E+00	.255E+02	.153E+09
349.20	.103E-02	-.271E+01	.717E+02	.461E+00	.323E+02	.153E+09
352.80	.123E-02	-.127E+01	.712E+02	.333E+00	.392E+02	.153E+09
356.40	.144E-02	.000E+00	.707E+02	.000E+00	.462E+02	.153E+09
360.00	.165E-02	.000E+00	.707E+02	.000E+00	.534E+02	.153E+09

COMPUTED LATERAL FORCE AT PILE HEAD = .40000E+02 KIP
COMPUTED MOMENT AT PILE HEAD = .00000E+00 IN-KIP
COMPUTED SLOPE AT PILE HEAD = -.37926E-02

THE OVERALL MOMENT IMBALANCE = .519E-08 IN-KIP
THE OVERALL LATERAL FORCE IMBALANCE = .885E-07 LBS

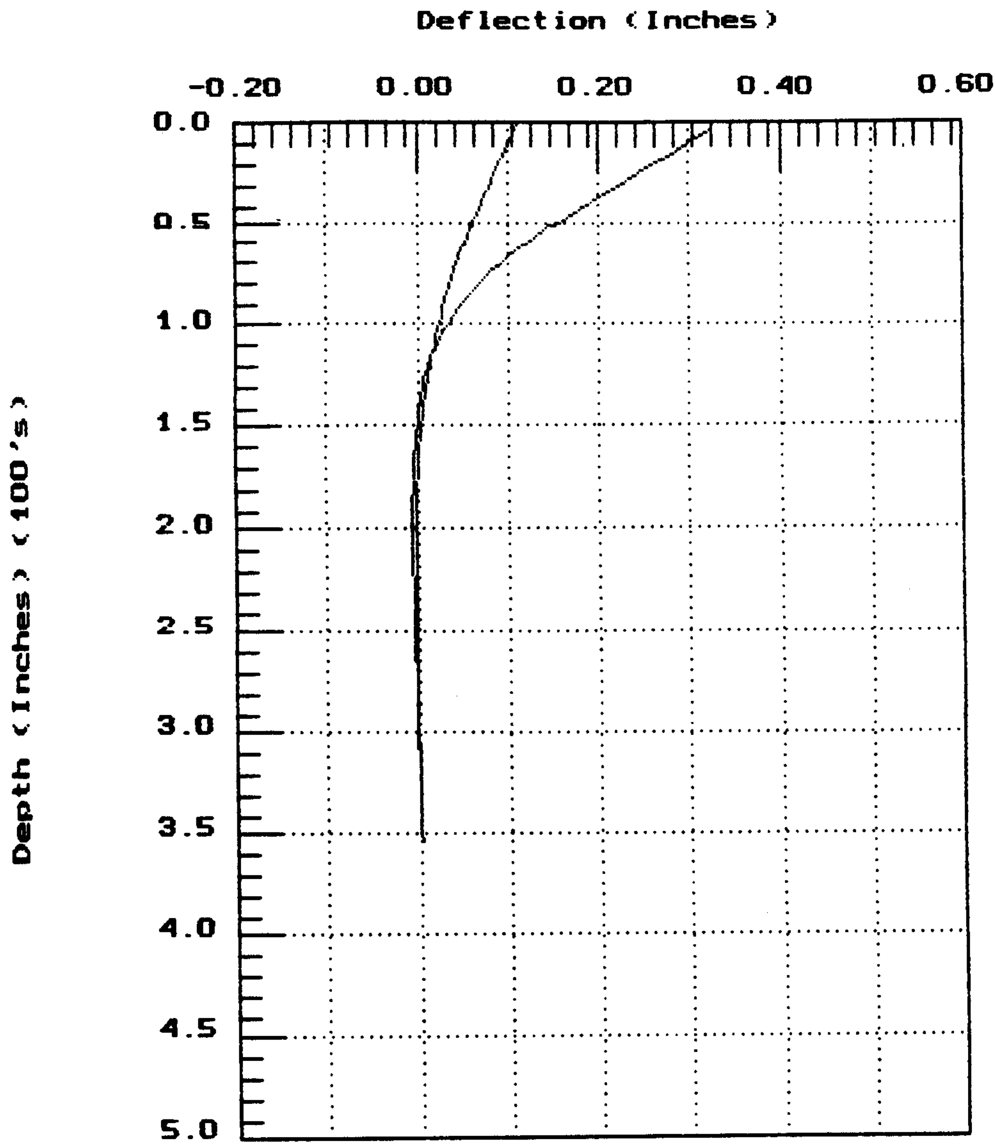
OUTPUT SUMMARY

PILE HEAD DEFLECTION = .337E+00 IN
MAXIMUM BENDING MOMENT = .217E+04 IN-KIP
MAXIMUM TOTAL STRESS = .890E+03 LBS/IN**2

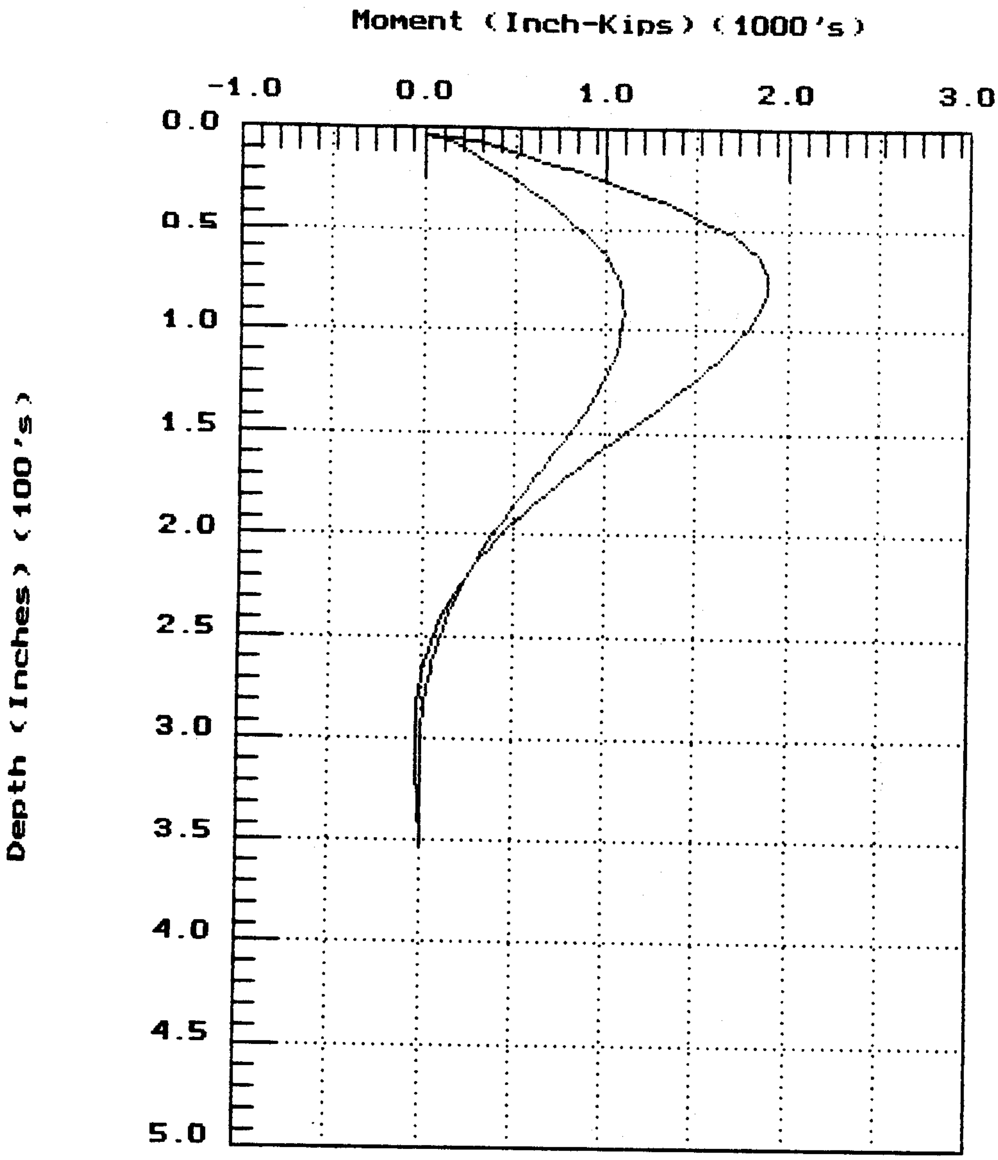
NO. OF ITERATIONS = 13
MAXIMUM DEFLECTION ERROR = .435E-05 IN

S U M M A R Y   T A B L E
****

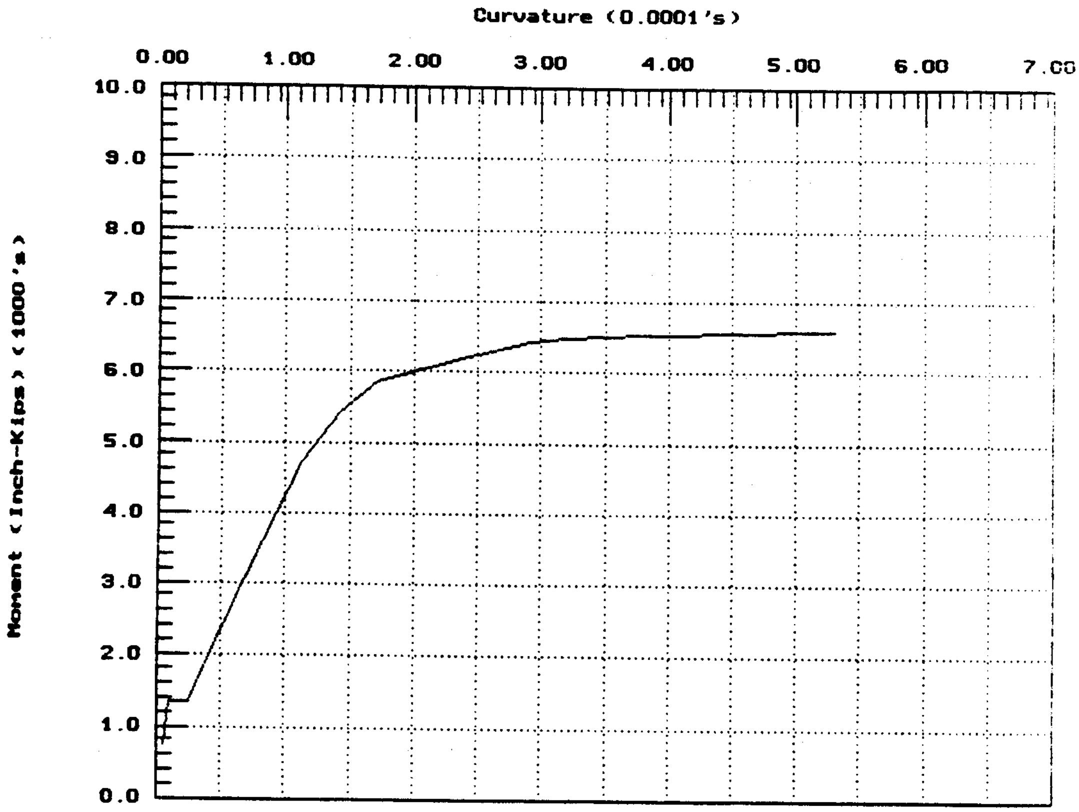
LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-KIP)	MAX. STRESS (LBS/IN**2)
.200E+02	.000E+00	.500E+02	.110E+00	-.105E-02	.112E+04	.493E+03
.400E+02	.000E+00	.500E+02	.337E+00	-.379E-02	.217E+04	.890E+03



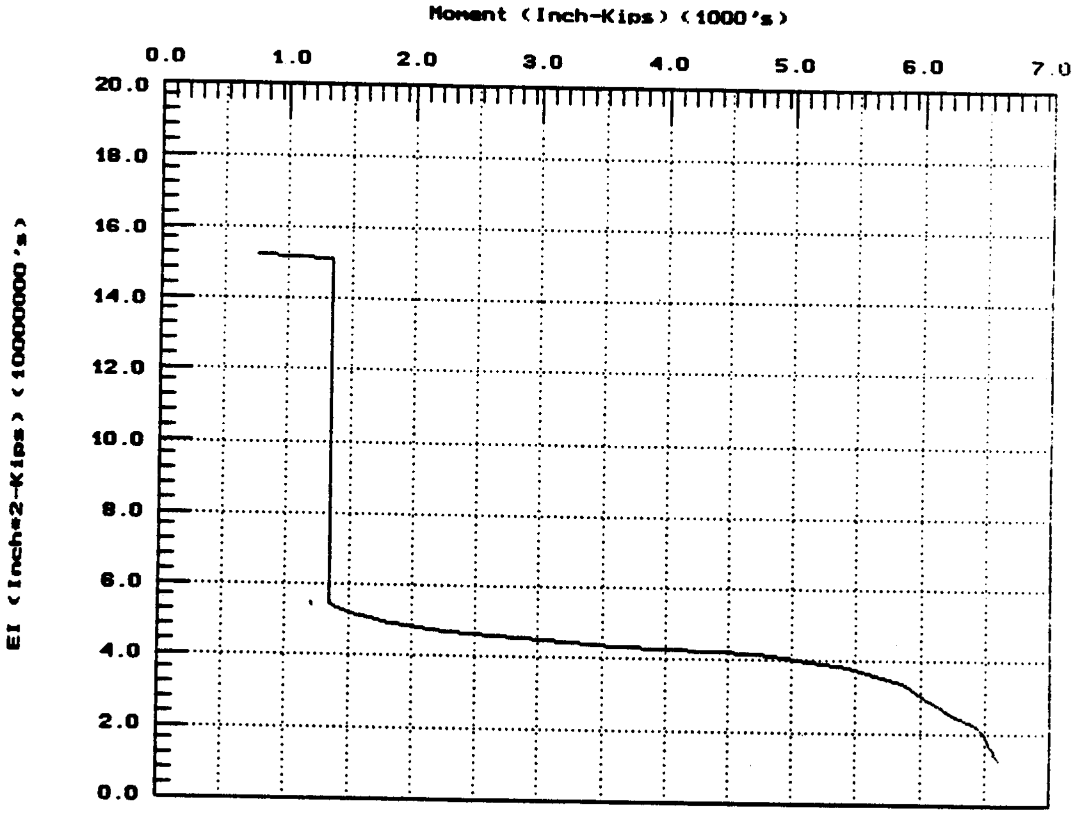
Deflection vs. Depth



Bending Moment vs. Depth



Bending Moment vs. Curvature



EI vs. Bending Moment

COM624P
LATERALLY LOADED PILE ANALYSIS PROGRAM FOR THE MICROCOMPUTER
Version 2.0

Part II: Background

CHAPTER 1. INTRODUCTION

The documentation for Computer Program COM624P consists of three documents: Part I, Users Guide; Part II Engineering Background; and Part III, Systems Maintenance.

The information shown in this document is limited to that needed for the operation of the computer program and to a brief introduction of the nature of the method of analysis. The user is referred to two documents published by the Federal Highway Administration for a relatively complete treatment of the topic (FHWA-IP-84-11 and FHWA/RD-85/106). A study of those publications and some of the papers that are cited therein will be necessary for the engineer to make proper use of COM624P. This program does not provide an "automatic" solution to the problem of the pile under lateral loading; rather, decisions of an experienced engineer are required in the selection of appropriate input and in the analysis of output of the program.

NATURE OF THE PROBLEM

The analysis of a pile under lateral loading is a problem in soil-structure interaction; that is, the deflection of the pile is dependent on the soil response and the soil response is a function of pile deflection. Thus, the problem cannot be solved by the equations of static equilibrium, but a differential equation must be solved to obtain the deflection of the pile. Iteration must be employed because the soil response is a nonlinear function of pile deflection and of position along the length of the pile.

Definition of Soil Response

The definition of soil response is given in Fig. 1.1. Figure 1.1a is an elevation view of a section of a pile with the depth identified at which the soil response is investigated. Figure 1.1b gives the distribution of unit stresses around the pile after its installation and before load is applied; if the pile has been installed without bending, there is no unbalanced force acting. If the pile is caused to deflect through a distance y_1 (exaggerated here for clarity of presentation), the unit stresses may be as shown in Fig. 1.1c. The unit stress has decreased on the back side of the pile and has increased on the front side. The unbalanced force is now p_1, in units of force per unit of length along the pile, and can be found by integrating the unit stresses.

A nonlinear relationship exists between p and y because, at some deflection y, the soil response p will reach a limit and remain constant, or perhaps decrease, with further deflection. The nonlinear curve relating the soil response and the pile deflection is termed a p-y curve. A family of p-y curves can be generated by methods discussed later and it is evident that the curves can vary in any arbitrary manner along the length of the pile.

Definition of Soil Modulus

The soil modulus, as employed in the solution of the laterally-loaded pile, is defined as p divided by y, has the units of force per length squared, and is given the symbol E_s. Thus, E_s can be characterized as a nonlinear spring whose stiffness is largest with small deflection and decreases as the deflection of the pile increases.

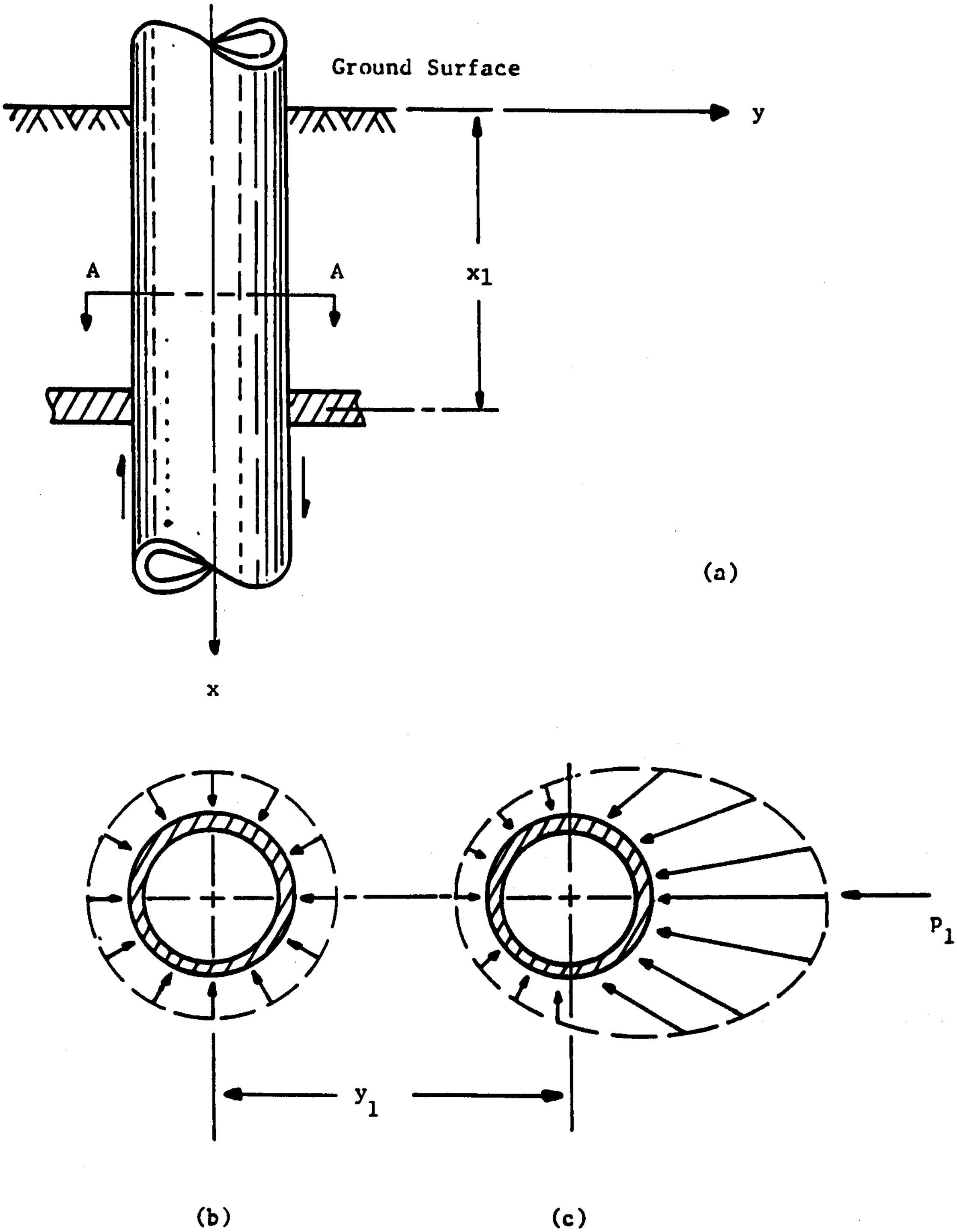


Figure 1.1. Definition of p and y (after Reese, 1983).

The left-hand portion of Fig. 1.2 shows the upper portion of a pile with three p-y curves in conceptual form. The curves are plotted in the second and fourth quadrants because the soil response is opposite in sign to pile deflection. The pile is assumed to be subjected to a lateral load P_t at its top. Dashed lines are drawn to show the possible deflection of the pile under two different loadings with the curvature in the deflection being ignored for ease in presentation. For each of the p-y curves, dotted lines are drawn to the intersection of the deflection with the p-y curve. The slope of these dotted lines indicates the magnitude of the soil modulus E_s at each of the particular points along the pile and at each particular deflection of the pile.

The right-hand portion of the sketch shows a plot of E_s as a function of distance x along the pile. As may be seen, E_s is some arbitrary function of x and y for any particular lateral load P_t at the top of the pile. The figure suggests that: a solution of the problem cannot in general be based on some presumed variation of E_s as a function of x; that E_s is not a property of the soil but is merely a fitting function to be determined; that the ability to formulate p-y curves is essential to a solution; that iteration will invariably be necessary; and that a computer program is essential.

A method of analysis employed in some engineering offices starts with the selection of a depth below the groundline at which the pile is assumed to be fixed against rotation. No soil is assumed to exist along the pile above that point; thus, the pile in soil is replaced by a cantilever beam, and solving for deflection and bending moment proceeds by using the ordinary equations of mechanics. If, by chance, the depth to the point of fixity was selected correctly, the computed maximum bending moment would agree with the actual maximum moment, but the distribution of bending moment along the pile would certainly be incorrect.

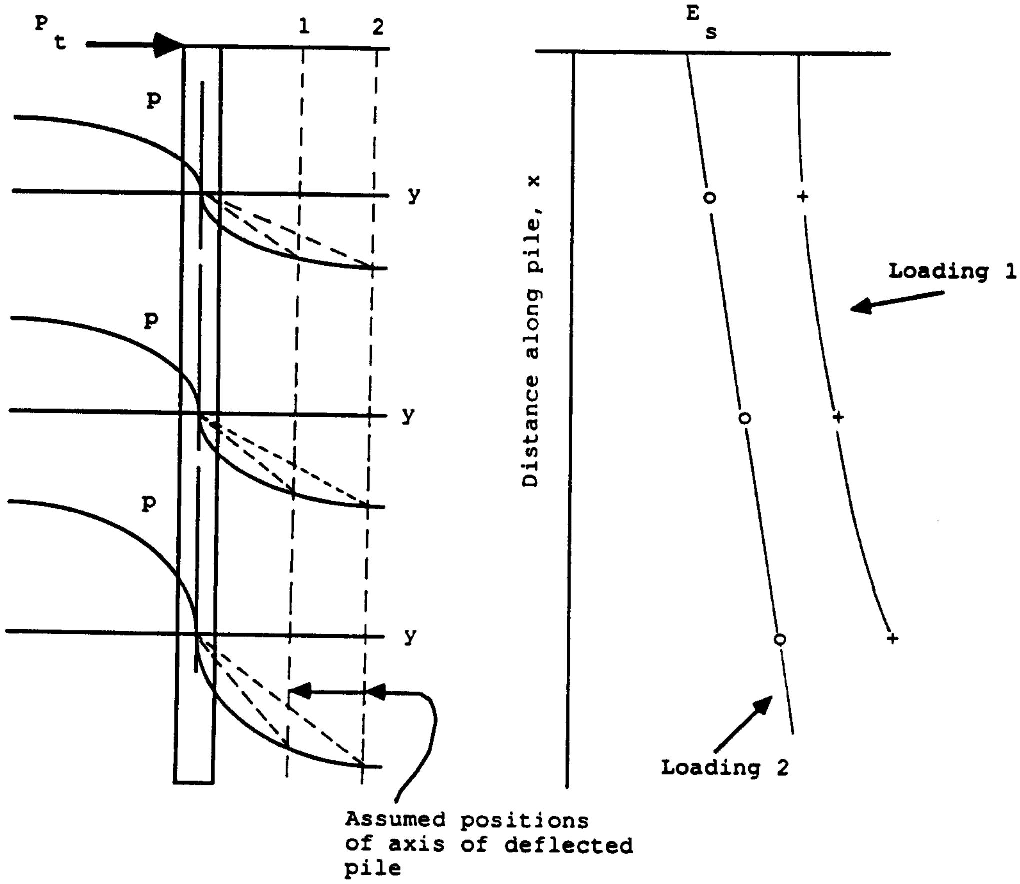


Figure 1.2. Concept of determination of soil modulus as a function of distance along pile.

Furthermore, the selection of a point of fixity (such that both the maximum deflection and the maximum bending both were computed correctly) would be a virtual impossibility. Thus, no guidelines can be developed for selecting a point of fixity that would allow the response of a pile to be computed accurately. This discussion is for the purpose of reinforcing the desirability of using the p-y method of analysis as presented herein.

DESIGN BY FACTORING THE LOAD

A pile under lateral load, and some amount of axial load as well, must be selected so that it has an appropriate factor of safety against collapse due to bending and against excessive deflection. If a curve were to be developed for bending moment or deflection, the curve would be nonlinear with respect to the lateral load. Thus, the preferred method of design of the pile is to find the factored load that will cause the pile to "fail." The factor is selected so as to provide an appropriate factor of safety with respect to load. If the allowable-stress approach is used, the load that produced the allowable stress could be increased only a small percentage and failure might occur because of the response of the nonlinear p-y curves.

The load-factor approach requires that the cross section of the pile be analyzed to determine the ultimate moment that will cause the development of a plastic hinge. Such values are tabulated for structural shapes, and computer programs are available to analyze composite sections, such as a reinforced-concrete section. The determination of some magnitude of deflection that will cause a failure is less straightforward. There may be some structures that are sensitive to deflection for a site-specific reason, and the load-factor method can be used to reduce the load that results in excessive deflection to a safe lateral load.

Another type of failure can be investigated by COM624P. The equation that is programmed and described later defines the behavior of a beam-column so that one of the input parameters is axial load. Some piles may extend some distance above the groundline so that buckling may be a problem. The failure of the pile in buckling can be investigated by holding the factored lateral load constant and by increasing the axial load in increments until the deflection becomes excessive. It is important that the axial load be increased in small increments because the procedure that is employed behaves erratically at loads above the buckling load.

NATURE OF LOADING

In respect to lateral loading, four kinds of loads are encountered in practice: short-term, repeated, sustained, and seismic or dynamic. The engineer must select the soil-response curves to be used in a particular design by giving careful consideration to the nature of the loading.

Static Loading

Short-term or static loading is frequently employed in a field test in order to obtain the response of a soil that can be correlated with the engineering properties of the soil. The p-y curves for static loading, thus, are a sort of "backbone" response by which the response of a pile to other sorts of loading can be judged. In some instances, the static p-y curves can be used in design. Methods of predicting p-y curves for static loading are presented later.

Cyclic Loading

Many structures are subjected to cyclic or repeated lateral loads. Wind gusts are an example. Other examples are traffic loads on curved bridges, braking loads, current and wave loads, and ice loads. The p-y curves that are proposed for cyclic loading are presented in a later section of this report. The proposals are strongly based on field experiments. Only a limited number of such experiments have been performed and the judgment of an experienced engineer is needed in ascertaining the magnitude of the load factor that is appropriate. In some instances, field load tests at the specific site are indicated.

Sustained Loading

Retaining walls, bulkheads, and bridge abutments are subjected to sustained loading. A pile in granular soil or heavily overconsolidated clay can be expected to undergo only a small amount of additional deflection, or perhaps none, depending on the magnitude of the unit stresses that exist around the pile. On the other hand, if the pile is installed in normally consolidated or lightly overconsolidated clay, the time-related deflection due to consolidation and creep may be significant. In concept, an analysis could be accomplished by stretching the y-values on the p-y curves an amount to accommodate the time-related displacement. However, no analytical method has been proposed for making the adjustments in the p-y curves.

The procedure that is suggested is to refer to any information that may be found in the technical literature; for example to the papers by Neukirchner and Nixon (1987), and Neukirchner (1987). Also, the computer program can be utilized to obtain an estimate under the working load of the lateral stresses against the soil. The theory of consolidation can be employed to gain some insight into the possible additional, time-related deflection of the pile. In this connection, some consideration must be given to the time-related changes in the soil stresses.

An alternative procedure in important cases is to install a test pile and subject it to sustained loading. The length of time the load can be maintained would be limited, of course; however, the additional deflection will decay exponentially so that it would be possible to make an extrapolation to estimate the final amount of additional deflection.

Dynamic or Seismic Loading

There may be instances in the design of piles where the lateral loading arises from vibration as from oscillating machinery. Because the deflections of a pile would, in general, be quite small due to the vibration, a constant value of soil modulus as a function of deflection could be selected. The reader is referred to technical literature on soil dynamics for guidance. With regard to the response of the pile, inertia effects cannot be ignored as is possible with static or with most cyclic loads.

A discussion of the design of a pile to sustain lateral loading that could be generated by an earthquake is beyond the scope of this report. The design may be made by a pseudo-static method or, if a rational method is to be employed, the analysis would start with an estimation of the free-field motion of the surface soils at the site.

INTERACTION OF THE PILES WITH THE SUPERSTRUCTURE

As shown later, the user of the program can select a variety of sets of boundary conditions at the top of the pile. The conditions of equilibrium and compatibility are satisfied by the appropriate selection. If a pile extends upward to support a road sign, the two boundary conditions consist of a shear and a moment. If a pile extends upward to form a part of the superstructure, the two boundary conditions consist of a shear and a rotational restraint. In order to select the proper magnitude of the rotational restraint, iteration between the pile foundation and the superstructure is usually necessary.

If a pile extends upward and is embedded in a concrete mat such as the base of a retaining wall, an acceptable solution in some cases is to assume that the pile head is fully fixed against rotation. The second boundary condition, the shear, may be selected by dividing the total lateral load of the wall by the number of piles. There may be occasions when the deflection at the pile head is one of the known boundary conditions. For example, a bridge may be constructed in such a way that the lateral deflection of the pile head is limited to a known amount.

In any case, the engineer must make a careful study of the manner in which the piles interact with the entire structure so that the proper input to the program can be selected. A number of trials may be necessary on occasion.

INFLUENCE OF PILE LENGTH

The length of the pile that is employed in the analyses by computer is an important consideration. An examination of the output for the solution of the behavior of a long pile will show that the deflection oscillates back and forth about the axis of the lower portion of the pile so that there are a large number of points of zero deflection. If the length of the pile is arbitrarily shortened so that there are only two or three points of zero deflection, a comparison of the two sets of results will show that there is no difference in the groundline deflection or in the magnitude of the maximum moment. As a matter of fact, there is no discernible difference in the two solutions for the portion of the pile above the first point of zero deflection. Therefore, the engineer may wish to shorten the length of a pile that is being analyzed in order to save computer time. This can be done by examining the results of the first run to discover the number of points of zero deflection. The length of the pile can then be shortened so that there are two or three points of zero deflection.

On the other hand, if the total length of a pile is not selected on the basis of axial loading but only on lateral loading, it will be desirable to make a series of computer runs with variation in the penetration of the pile. As shown in Fig. 1.3, a critical penetration will be found. At penetrations less than the critical, the top of the pile will experience additional deflection because the bottom of the pile is deflecting. The pile is undergoing a "fence-posting" action, a condition that is generally undesirable.

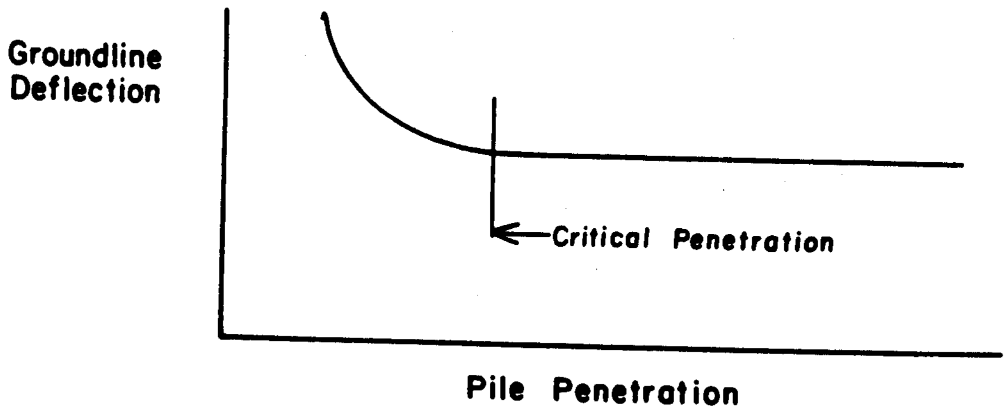


Figure 1.3. Influence of pile penetration on groundline deflection.

PILE GROUPS UNDER LATERAL LOADING

Single piles can be employed to support an overhead sign but most piles are installed in groups. Two problems must be addressed with respect to pile groups: the loss of efficiency due to close spacing, and the distribution of the load from the superstructure to each of the supporting piles.

The second of these problems can be solved rationally if the three nonlinear stiffnesses at the pile head; for axial load, for lateral load, and for moment; can be defined. The result is the vertical displacement, lateral displacement, and rotation of the superstructure and the corresponding movements at each of the pile heads. The response of each of the piles is computed. The solution is as accurate as the pile head stiffnesses can be determined.

The problem of the interaction between groups of closely spaced piles cannot be solved with the same assurance as can the one described above. The theory of elasticity has been employed to develop interaction factors, but experiments have shown that these factors can be seriously in error. Other methods have been suggested but research has not yet developed sufficient information to allow an engineer to make a confident prediction. The behavior of piles in groups is discussed in the two FHWA publications on laterally loaded piles mentioned earlier (FHWA, 1984; FHWA, 1986).

VERIFICATION OF ACCURACY OF THE COMPUTER OUTPUT

Chapter 5 of this report is concerned in some detail with the verification of the results of the computations. An important consideration is that the results should be considered as questionable until the engineer has done an independent study, however brief, to verify the solution.

The computer will produce results in a short period of time that would take weeks, or much longer, with the calculator. Deflection, rotation, bending moment, shear, and soil resistance are given point by point along the length of a pile, and the equations of equilibrium and compatibility are automatically satisfied. A series of loads can be input and the computer will rapidly produce pile-head deflection and maximum bending moment as a function of load. If desired, the results can be readily displayed in graphical form.

The versatility and utility of the computer program are so impressive that it is difficult not to accept the results as correct; however, the engineer is well advised to question the results and to adopt some routine means for verification. The methods that are indicated in Chapter 5 should prove helpful and the engineer may devise other methods that are applicable to local situations.

CHAPTER 2. BASIC THEORY OF COMPUTATION

THE DIFFERENTIAL EQUATION

The standard differential equation for the deflection of a beam as presented in textbooks on mechanics provides the basis for the analysis of most of the cases of piles under lateral loading. The only adjustment that is needed to the basic equation is to replace the distributed load p with the soil modulus E_s times the pile deflection y (with a negative sign).

However, if the axial load is relatively large or if an unsupported portion of the pile extends above the groundline, the inclusion of the effect of the axial load in the differential equation is necessary. The beam-column equation that is derived can be used to investigate buckling and, for cases where the axial load is applied at the groundline, will allow the additional lateral deflection due to axial loading to be computed.

The derivation of the beam-column was done by Hetenyi (1946). The pile is assumed to be replaced by a bar and a segment, bounded by two horizontals a distance of dx apart, has been cut from the bar as shown in Fig. 2.1. The segment has been displaced due to lateral loading and a pair of vertical compressive forces P_x are acting at the center of gravity of the end cross sections of the bar.

The equilibrium of moments (ignoring second-order terms) leads to the equation:

$$(M + dM) - M + P_x dy - V_v dx = 0 \qquad (2.1)$$

or

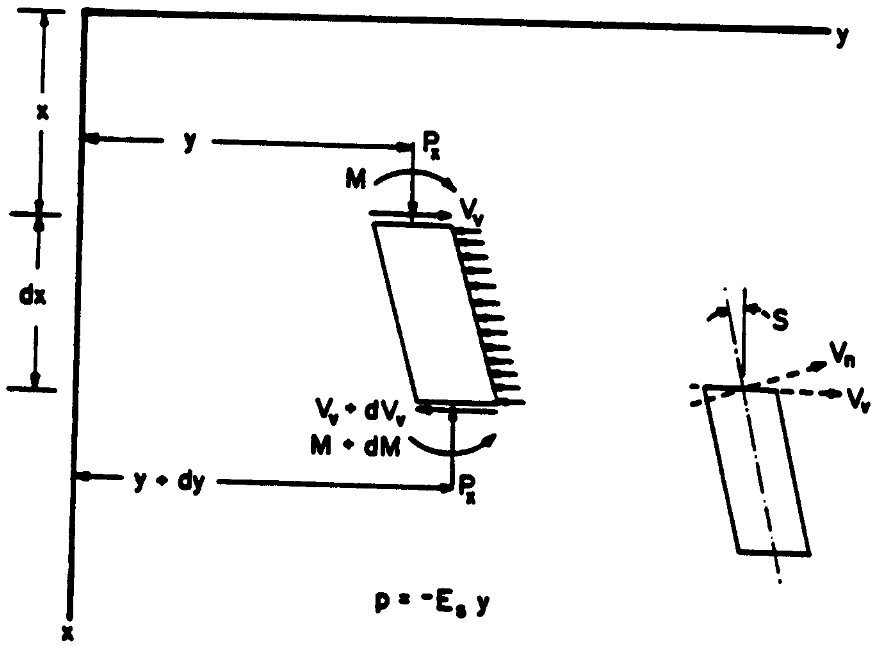


Figure 2.1. Element for beam-column (after Hetenyi, 1946).

$$\frac{dM}{dx} + P_x \frac{dy}{dx} - V_v = 0. \tag{2.2}$$

Differentiating Eq. 2.2 with respect to x, the following equation is obtained:

$$\frac{d^2M}{dx^2} + P_x \frac{d^2y}{dx^2} - \frac{dV_v}{dx} = 0. \tag{2.3}$$

The following identities are noted:

$$\frac{d^2M}{dx^2} = EI \frac{d^4y}{dx^4}, \tag{2.4}$$

$$\frac{dV_v}{dx} = p, \text{ and} \tag{2.5}$$

$$p = -E_s y. \tag{2.6}$$

And making the indicated substitutions, Eq. 2.3 becomes:

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} + E_s y = 0. \tag{2.7}$$

The direction of the shearing force V_v is shown in Fig. 2.1. The shearing force in the plane normal to the deflection line can be obtained as:

$$V_n = V_v \cos S - P_x \sin S. \tag{2.8}$$

Because S is usually small, cos S = 1 and sin S = tan S = $\frac{dy}{dx}$. Thus, Eq. 2.9 is obtained:

$$V_n = V_v - P_x \frac{dy}{dx}. \tag{2.9}$$

V_n will mostly be used in computations but V_v can be computed from Eq. 2.9 where dy/dx is equal to the rotation S.

The following assumptions are made in deriving the differential equation:

- the pile has a longitudinal plane of symmetry; loads and reactions lie in that plane,
- the modulus of elasticity of the pile material is the same for tension and compression,
- transverse deflections of the pile are small,
- the pile is not subjected to dynamic loading, and
- deflections due to shearing stresses are negligible.

The sign conventions that are employed are shown in Fig. 2.2. For ease of understanding, the sign conventions are presented for a beam that is oriented like a pile. A solution of the differential equation yields values of y as a function of x. A family of curves can then be obtained as shown in Fig. 2.3 by using the following basic equations:

$$EI \frac{d^3y}{dx^3} = V \tag{2.10}$$

$$EI \frac{d^2y}{dx^2} = M, \text{ and} \tag{2.11}$$

$$\frac{dy}{dx} = S, \tag{2.12}$$

where

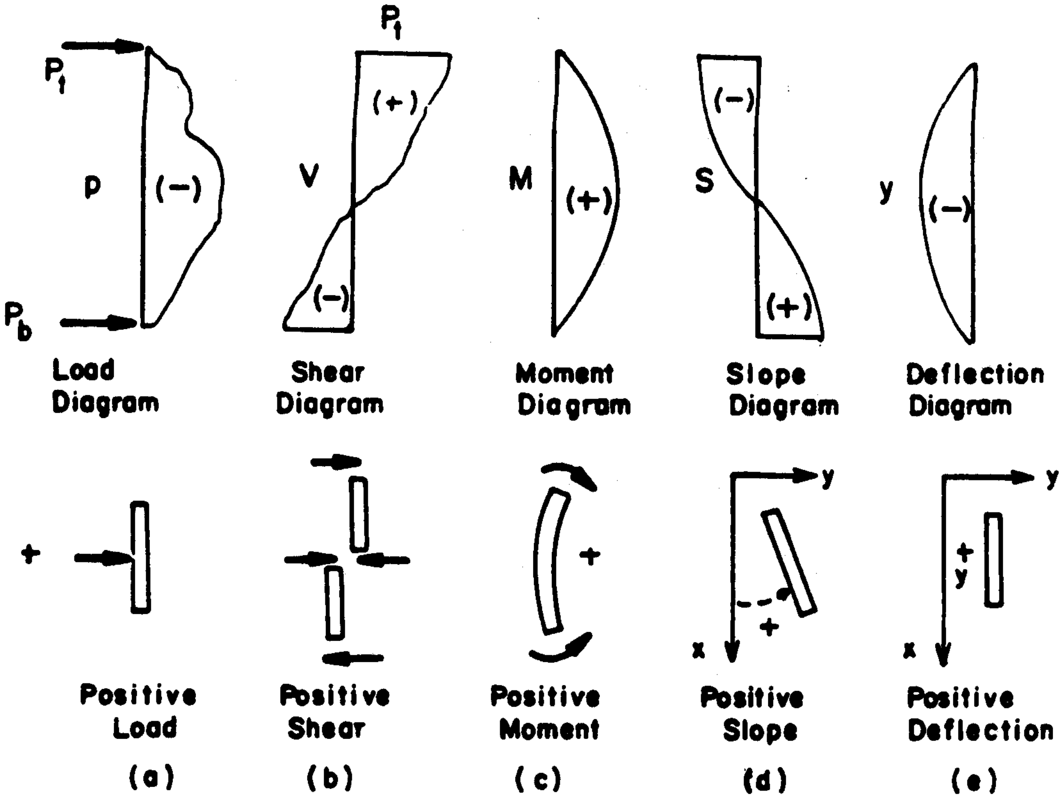


Figure 2.2. Sign conventions.

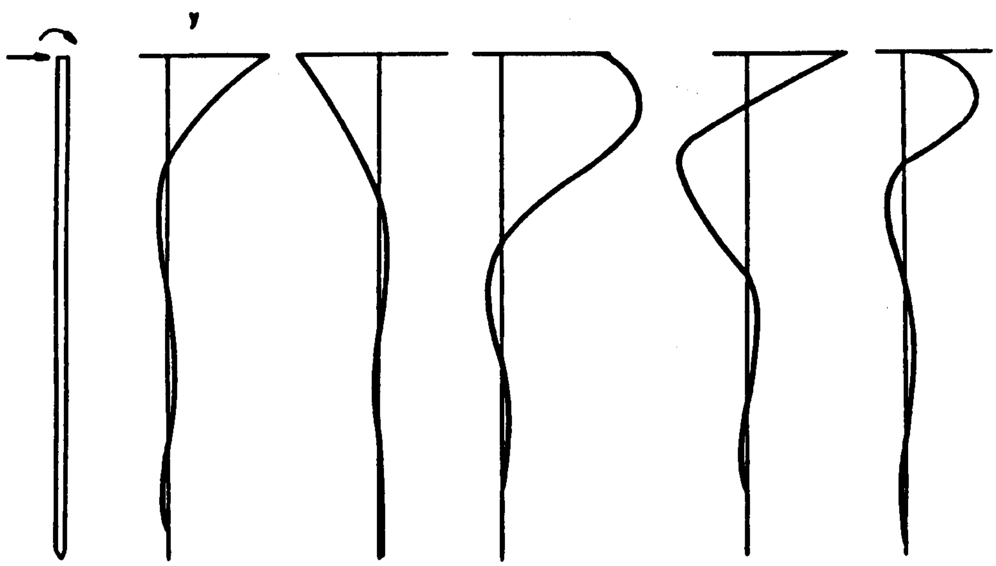


Figure 2.3. Form of the results obtained from a complete solution.

V = shear,

M = bending moment of the pile, and

S = slope of the elastic curve.

SOLUTION TO THE GOVERNING DIFFERENTIAL EQUATION

Equation 2.7 is rewritten and shown as Eq. 2.13. The term W, which is exactly similar to p, is added to allow a distributed load to be placed along the pile as, for example, when the pile extends above the groundline and is subjected to a distributed load from water currents or wind. The term k is substituted for E_s for ease in writing the equations.

$$\frac{d^2M}{dx^2} + P_x \frac{d^2y}{dx^2} + ky - W = 0 \qquad (2.13)$$

Equation 2.13 can be solved readily by using finite-difference techniques. The deflection of the pile by finite deflections is shown in Fig. 2.4. The finite difference expressions for the first two terms of Eq. 2.13 at point m are:

$$\left(\frac{d^2M}{dx^2}\right)_m = \Big[y_{m-2} R_{m-1} + y_{m-1} (-2R_m - 2R_{m-1})$$

$$+ y_m (4R_m + R_{m-1} + R_{m+1})$$

$$+ y_{m+1} (-2R_m - 2R_{m+1}) + y_{m+2}R_{m+1}\Big] \frac{1}{h^4}, \qquad (2.14)$$

$$P_x \left(\frac{d^2y}{dx^2}\right)_m = \frac{P_x (y_{m-1} - 2y_m + y_{m+1})}{h^2}, \qquad (2.15)$$

where

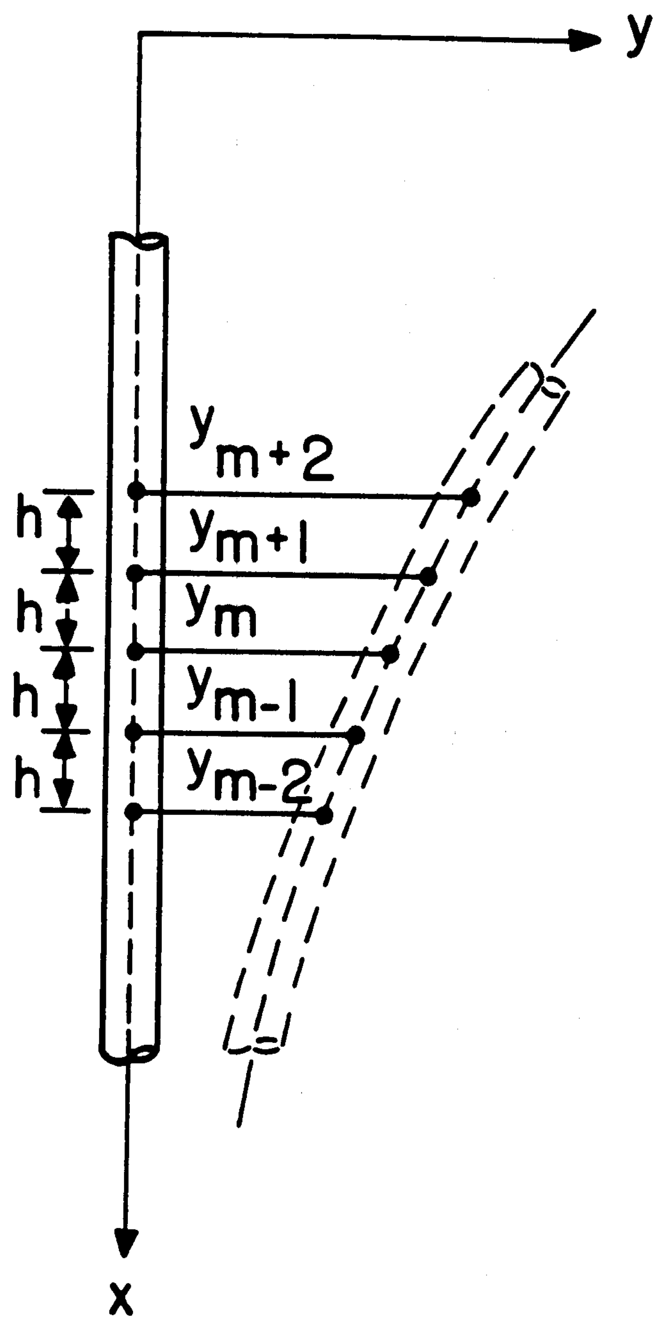


Figure 2.4. Representation of deflected pile.

R_m = flexural rigidity at point (m), that is,

$$R_m = E_m I_m. \qquad (2.16)$$

Equations 2.14 and 2.15 are substituted for terms in Eq. 2.13 and the resulting equation for Point m along the pile and Eq. 2.17 results.

$$y_{m-2} R_{m-1} + y_{m-1} \left(-2R_{m-1} - 2R_m + P_x h^2\right) + y_m \left(R_{m-1} + 4R_m + R_{m+1} - 2P_x h^2 + k_m h^4\right) + y_{m+1} \left(-2R_m - 2R_{m+1} + P_x h^2\right) + y_{m+2} R_{m+1} - W_m h^4 = 0. \qquad (2.17)$$

The axial force P_x which produces compression is assumed to be positive. Also, P_x acts through the axis of the pile; thus, P_x causes no moment at the top of the pile.

Applying the boundary conditions at the bottom of the pile, the solution to the differential equation in difference form can proceed (Gleser, 1953).

Using the notation shown in Fig. 2.5, the two boundary conditions at the bottom of the pile (point 0) are zero bending moment,

$$\left(\frac{d^2y}{dx^2}\right)_0 = 0, \qquad (2.18)$$

and zero shear,

$$R_0 \left(\frac{d^3y}{dx^3}\right)_0 + P_x \left(\frac{dy}{dx}\right)_0 = 0. \qquad (2.19)$$

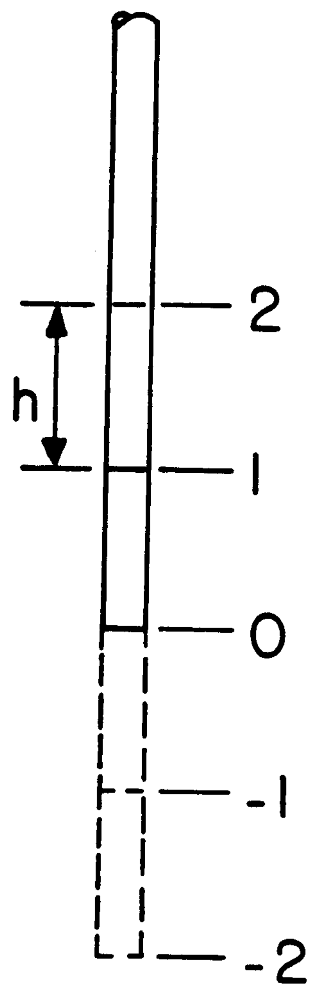


Figure 2.5. Points at bottom of pile.

For simplicity it is assumed that:

$$R_{-1} = R_0 = R_1. \tag{2.20}$$

These boundary conditions are, in finite difference form,

$$y_{-1} - 2y_0 + y_1 = 0, \text{ and} \tag{2.21}$$

$$y_{-2} = y_{-1}\left(2 - \frac{P_x h^2}{R_0}\right) - y_1\left(2 - \frac{P_x h^2}{R_0}\right) + y_2, \tag{2.22}$$

respectively. Substituting these boundary conditions in finite difference form in Eq. 2.17 where m is equal to zero, and rearranging terms, results in the following equations:

$$y_0 = a_0 y_1 - b_0 y_2 + d_0, \tag{2.23}$$

$$a_0 = \frac{2R_0 + 2R_1 - 2P_x h^2}{R_0 + R_1 + k_0 h^4 - 2P_x h^2}, \tag{2.24}$$

$$b_0 = \frac{R_0 + R_1}{R_0 + R_1 + k_0 h^4 - 2P_x h^2}, \text{ and} \tag{2.25}$$

$$d_0 = \frac{W_0 h^4}{R_0 + R_1 + k_0 h^4 - 2P_x h^2}. \tag{2.26}$$

Equation 2.17 can be expressed for all values of m other than 0 and the top of the pile by the following relationships:

$$y_m = a_m y_{m+1} - b_m y_{m+2} + d_m, \tag{2.27}$$

$$a_m = \frac{-2b_{m-1}R_{m-1}+a_{m-2}b_{m-1}R_{m-1}+2R_m-2b_{m-1}R_m+2R_{m+1}-P_x h^2 \left(1-b_{m-1}\right)}{c_m}, \tag{2.28}$$

$$b_m = \frac{R_{m+1}}{c_m}, \text{ and} \tag{2.29}$$

$$d_m = \frac{W_m h^4 - d_{m-1}\left(a_{m-2}R_{m-1}-2R_{m-1}-2R_m+P_x h^2\right) - d_{m-2}R_{m-1}}{c_m}. \tag{2.30}$$

where

$$c_m = R_{m-1} - 2a_{m-1}R_{m-1} - b_{m-2}R_{m-1} + a_{m-2}a_{m-1}R_{m-1}$$

$$+ 4R_m - 2a_{m-1} R_m + R_{m+1} + k_m h^4 - P_x h^2 \; (2-a_{m-1}) \tag{2.31}$$

The top of the pile (m=t) is shown in Fig. 2.6. Four sets of boundary conditions are considered. These are designated as Cases 1 through 4.

1. The lateral load (P_t) and the moment (M_t) at the top of the pile are known.

2. The lateral load (P_t) and the slope of the elastic curve (S_t) at the top of the pile are known.

3. The lateral load (P_t) and the rotational-restraint constant (M_t/S_t) at the top of the pile are known.

4. The moment (M_t) and the deflection (y_t) at the top of the pile are known.

For convenience in establishing expressions for these boundary conditions, the following constants are defined:

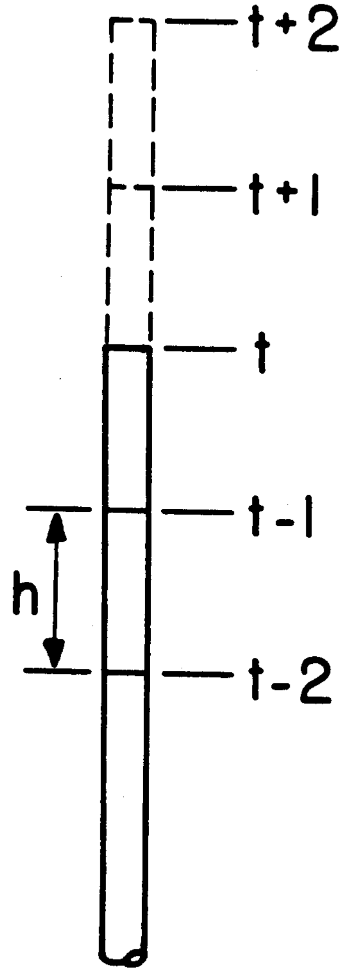


Figure 2.6. Points at top of pile.

$$J_1 = 2hS_t, \tag{2.32}$$

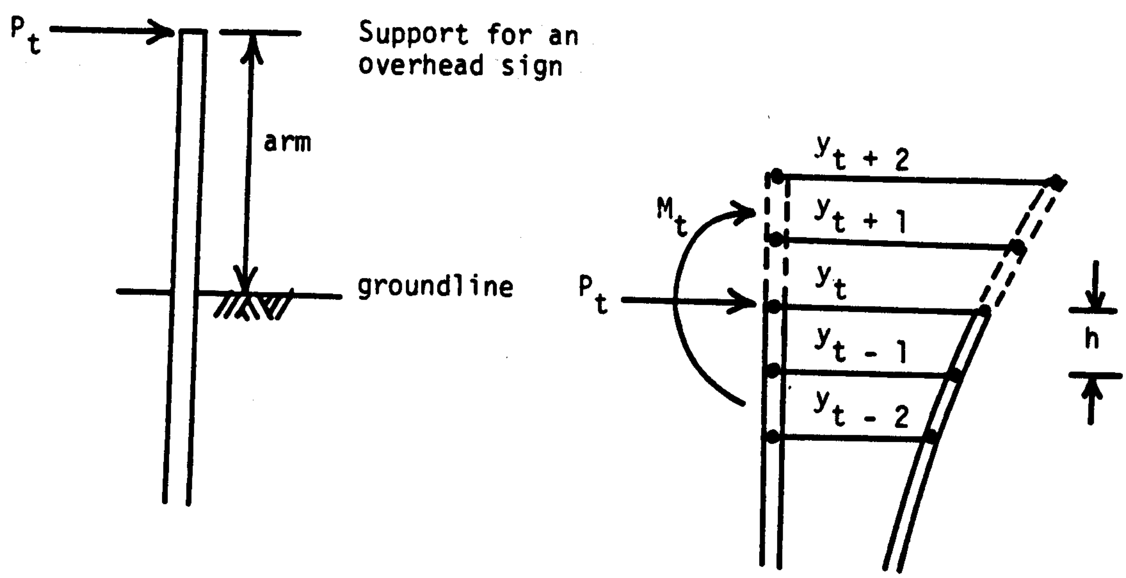
$$J_2 = \frac{M_t h^2}{R_t}, \tag{2.33}$$

$$J_3 = \frac{2P_t h^3}{R_t}, \tag{2.34}$$

$$J_4 = \frac{h}{2R_t} \frac{M_t}{S_t}, \text{ and} \tag{2.35}$$

$$E = \frac{-P_x h^2}{R_t}. \tag{2.36}$$

The boundary conditions for Case 1 are shown by the sketches in Fig. 2.7. The difference equations for the top of the pile are as shown in Eqs. 2.37 and 2.38.



Note: P_t and M_t are known; they are shown in the positive sense in the sketches.

Figure 2.7. Case 1 of boundary conditions at top of pile.

$$\frac{R_t}{2h^3}\left(y_{t-2} - 2y_{t-1} + 2y_{t+1} - y_{t+2}\right)$$

$$+ \frac{P_x}{2h}\left(y_{t-1} - y_{t+1}\right) = P_t, \text{ and} \qquad (2.37)$$

$$\frac{R_t}{h^2}\left(y_{t-1} - 2y_t + y_{t+1}\right) = M_t. \qquad (2.38)$$

After substitutions the difference equations for the deflection at the top of the pile and at the two imaginary points above the top of the pile are:

$$y_t = \frac{Q_2}{Q_1}, \qquad (2.39)$$

$$y_{t+1} = \frac{J_2 + G_1 y_t - d_{t-1}}{G_2}, \text{ and} \qquad (2.40)$$

$$y_{t+2} = \frac{a_t y_{t+1} - y_t + d_t}{b_t}, \qquad (2.41)$$

where

$$Q_1 = H_1 + \frac{G_1 H_2}{G_2} + \left(1 - a_t \frac{G_1}{G_2}\right)\frac{1}{b_t}, \qquad (2.42)$$

$$Q_2 = J_3 + \frac{a_t\left(J_2 - d_{t-1}\right)}{b_t G_2} + \frac{H_2\left(d_{t-1} - J_2\right)}{G_2} + \frac{d_t}{b_t}$$

$$+ d_{t-1}\left(2 + E - a_{t-2}\right) - d_{t-2}, \qquad (2.43)$$

$$G_1 = 2 - a_{t-1}, \qquad (2.44)$$

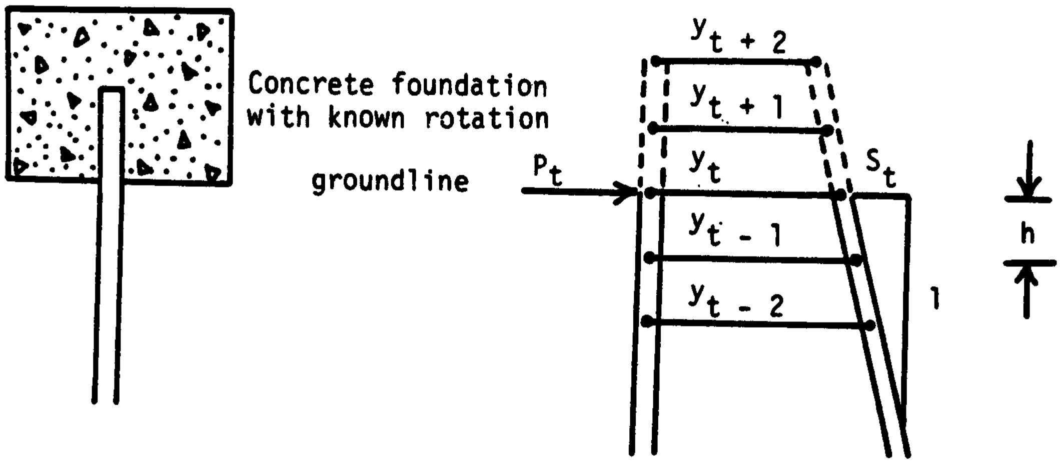
$$G_2 = 1 - b_{t-1}, \qquad (2.45)$$

$$H_1 = -2a_{t-1} - Ea_{t-1} - b_{t-2} + a_{t-1}a_{t-2}, \text{ and} \qquad (2.46)$$

$$H_2 = -a_{t-2}b_{t-1} + 2b_{t-1} + 2 + E\left(1 + b_{t-1}\right). \qquad (2.47)$$

The boundary conditions for Case 2 are shown by the sketches in Fig. 2.8. The difference equations for the boundary conditions are given by Eq. 2.37 given earlier and Eq. 2.48 shown below.

$$y_{t-1} - y_{t+1} = J_1 \qquad (2.48)$$



Note: P_t and S_t are known; they are shown in the positive sense.

Figure 2.8. Case 2 of boundary conditions at top of pile.

The resulting difference equations for the deflections at the three points at the top of the pile are:

$$y_t = \frac{Q_4}{Q_3}, \qquad (2.49)$$

$$y_{t+1} = \frac{a_{t-1}y_t - J_1 + d_{t-1}}{G_4}, \text{ and} \qquad (2.50)$$

$$y_{t+2} = \frac{a_t y_{t+1} - y_t + d_t}{b_t}, \tag{2.51}$$

where

$$Q_3 = H_1 + \frac{H_2 a_{t-1}}{G_4} - \frac{a_t a_{t-1}}{b_t G_4} + \frac{1}{b_t}, \tag{2.52}$$

$$Q_4 = J_3 + \frac{J_1 H_2}{G_4} - \frac{a_t (J_1 - d_{t-1}) - G_4 d_t + b_t d_{t-1} H_2}{b_t G_4}$$

$$+ d_{t-1} \left(2 + E - a_{t-2}\right) - d_{t-2}, \text{ and} \tag{2.53}$$

$$G_4 = 1 + b_{t-1}. \tag{2.54}$$

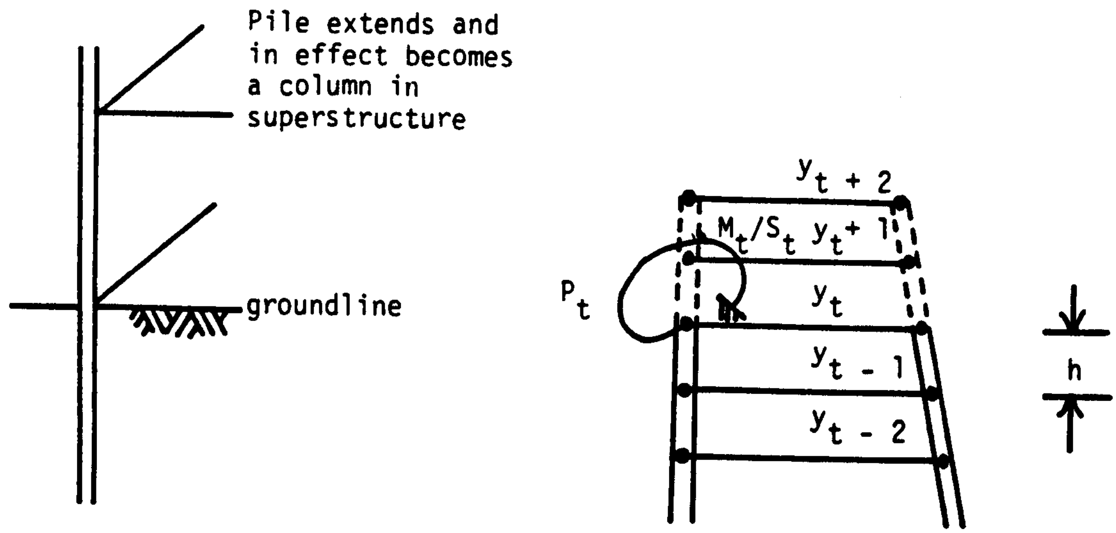
and the other constants are as previously defined.

The boundary conditions for Case 3 are shown by the sketches in Fig. 2.9. The difference equations for the boundary conditions are Eq. 2.37 given earlier and Eq. 2.55 shown below.

$$\frac{y_{t-1} - 2y_t + y_{t+1}}{y_{t-1} - y_{t+1}} = J_4 \tag{2.55}$$

The resulting difference equations for the deflections at the three points at the top of the pile are:

$$y_t = \frac{J_3 - \frac{a_t d_{t-1}(1-J_4)}{b_t (G_2 + J_4 G_4)} + \frac{d_t}{b_t} + d_{t-1}(2 + E - a_{t-2}) - d_{t-2} + \frac{d_{t-1} H_2 (1-J_4)}{G_2 + J_4 G_4}}{H_1 + H_2 H_3 - \frac{a_t}{b_t} H_3 + \frac{1}{b_t}}, \tag{2.56}$$



Note: P_t and M_t/S_t are known; they are shown in the positive sense in the sketches

Figure 2.9. Case 3 of boundary conditions at top of pile.

$$y_{t+1} = \frac{y_t(G_1+J_4a_{t-1}) - d_{t-1}(1-J_4)}{G_2 + J_4G_4} = H_3y_t - \frac{d_{t-1}(1-J_4)}{G_2 + J_4G_4}, \text{ and} \tag{2.57}$$

$$y_{t+2} = \frac{1}{b_t}\,(a_ty_{t+1} - y_t + d_t), \tag{2.58}$$

where

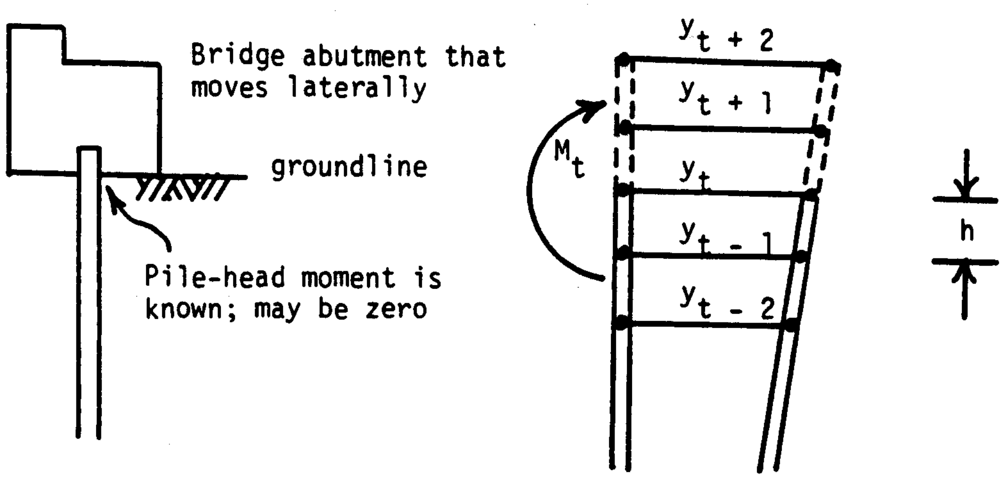
$$H_3 = \frac{G_1 + J_4a_{t-1}}{G_2 + J_4G_4}. \tag{2.59}$$

The other constants have been previously defined.

The boundary conditions for Case 4 are shown by the sketches in Fig. 2.10. The difference equations are given by Eq. 2.38 given earlier and by Eq. 2.60 given below.

$$y_t = y_t \qquad (2.60)$$

Using the above equations with a family of p-y curves, iteration is carried out until the solution converges to appropriate values of k at all points along the pile. Thus, the behavior of a pile under lateral load may be obtained by using COM624P.



Note: M_t and Y_t are known; they are shown in the positive sense in the sketches

Figure 2.10. Case 4 of boundary conditions at top of pile.

CHAPTER 3. SOIL RESPONSE CURVES (p-y CURVES)

INTRODUCTION

As noted earlier, the soil response is characterized as a set of discrete mechanisms such that the soil response at a point is not dependent on pile deflection elsewhere, thus, a continuum is not perfectly modeled. However, the recommendations for predicting p-y curves, as presented herein, are based on full-scale experiments in which the continuum effect was explicitly implemented. Furthermore, a small amount of unpublished experimental data suggests that the soil response at a point is unaffected by those changes in deflected shape that can be achieved by altering the rotational restraint at the pile head by any practical amount.

The three factors that have the most influence on a p-y curve are the soil properties, the pile geometry, and the nature of loading. The correlations that have been developed for predicting soil response have been based on the best estimate of the properties of the in-situ soil with no adjustment for the effects on soil properties of the method of installation. The logic supporting this approach is that the effects of pile installation on soil properties are principally confined to a zone of soil close to the pile wall, while a mass of soil of several diameters from the pile is stressed as lateral deflection occurs. There are instances, of course, where the method of pile installation must be considered; for example, if a pile is jetted into place, a considerable volume of soil could be removed with a significant effect on the soil response.

The principal dimension of a pile affecting the soil response is its diameter. All of the recommendations for developing p-y curves include a term for the diameter of the pile; if the cross section of the pile is not circular, the width of the pile perpendicular to the direction of loading is usually taken as the diameter.

USE OF SOIL MODELS TO DETERMINE SOIL BEHAVIOR

Some writers have made use of the theory of elasticity to develop expressions for p as a function of y, but the approach has been of limited use. Soil behavior can be modeled by the theory of elasticity only for very small strains. The limit-equilibrium approach applies at large strains and is employed herein to develop some useful expressions.

Soil Models for Saturated Clay

The assumed model for estimating the ultimate soil resistance near the ground surface is shown in Fig. 3.1 (Reese, 1958). The force F_p is

$$F_p = c_a bH \, [\tan \alpha + (1+K) \cot \alpha] + \frac{1}{2} \gamma b H^2 + c_a H^2 \sec \alpha \tag{3.1}$$

where

c_a = average drained shear strength,

K = a reduction factor to be multiplied by c_a to yield the average sliding stress between the pile and the stiff clay, and

γ = average unit weight of soil.
(the other terms are defined in the figure)

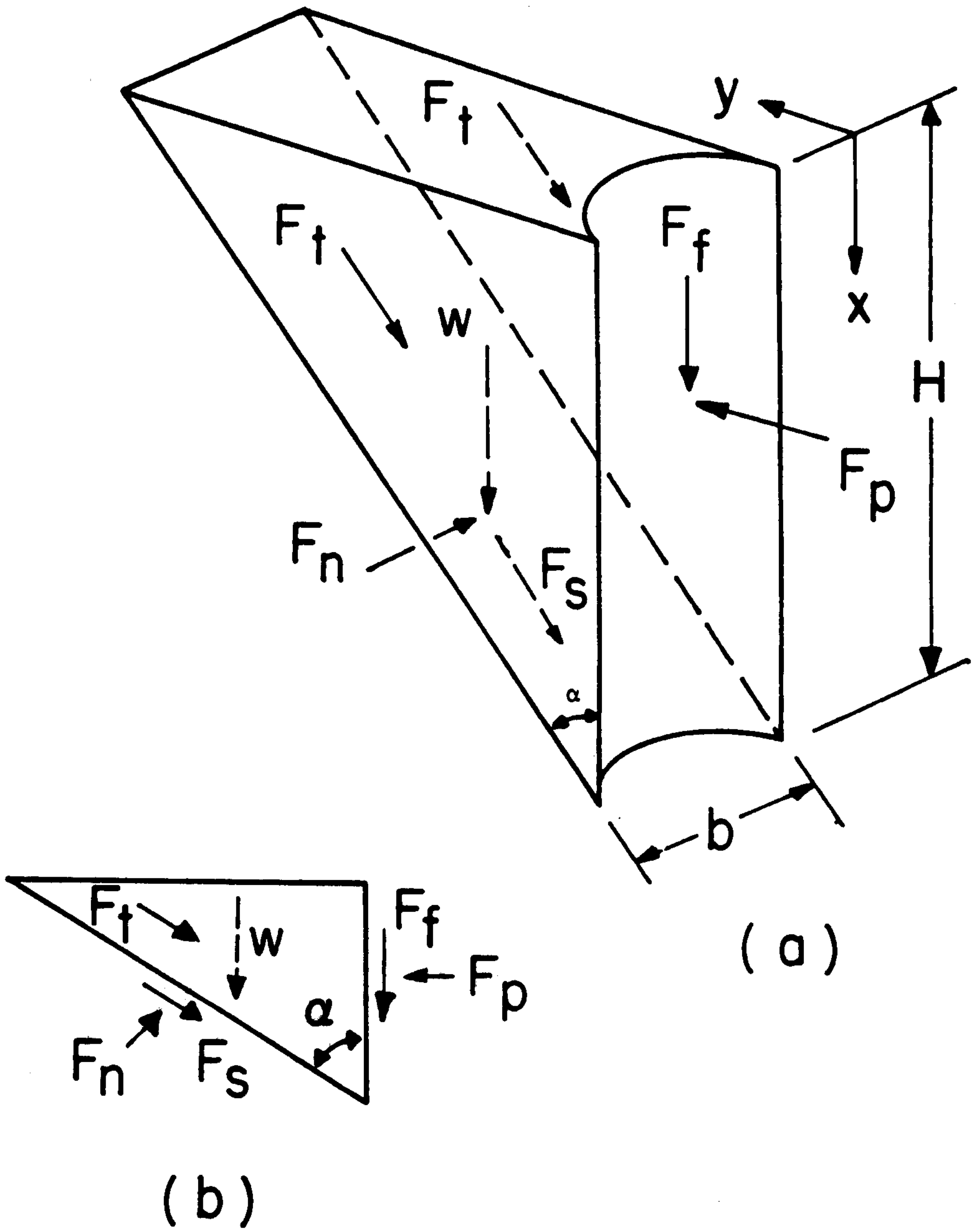


Figure 3.1. Assumed passive wedge-type failure for clay: (a) shape of wedge (b) forces acting on wedge (after Reese, 1958).

The angle α is taken as 45 degrees and K is assumed equal to zero. Differentiation of the resulting expression with respect to H yields an expression for the ultimate soil resistance as follows.

$$(p_u)_{ca} = 2c_a b + \gamma bH + 2.83\ c_a H \qquad (3.2)$$

It can be reasoned that at some distance below the ground surface the soil must flow around the deflected pile. The model for such movement is shown in Fig. 3.2a. If it is assumed that blocks 1, 2, 4, and 5 fail by shear and that block 3 develops resistance by sliding, the stress conditions are represented by Fig. 3.2b. By examining a free body of a section of the pile, Fig. 3.2c, one can conclude that:

$$(p_u)_{cb} = 11cb. \qquad (3.3)$$

Equations 3.2 and 3.3 are, of course, approximate but they do indicate the general form of the expressions that give the ultimate soil resistance along the pile. The equations can be solved simultaneously to find the depth at which the failure would change from the wedge-type to the flow-around type.

Soil Models for Sand

The soil model for computing the ultimate resistance near the ground surface is shown in Fig. 3.3a (Reese, Cox, and Koop, 1974). The total lateral force F_{pt} (Fig. 3.3c) may be computed by subtracting the active force F_a, computed using Rankine theory, from the passive force F_p, computed from the model. The force F_p is computed by assuming that the Mohr-Coulomb failure condition is satisfied on planes ADE, BCF, and AEFB. The directions of the forces are shown in Fig. 3.3b. No frictional force is assumed to

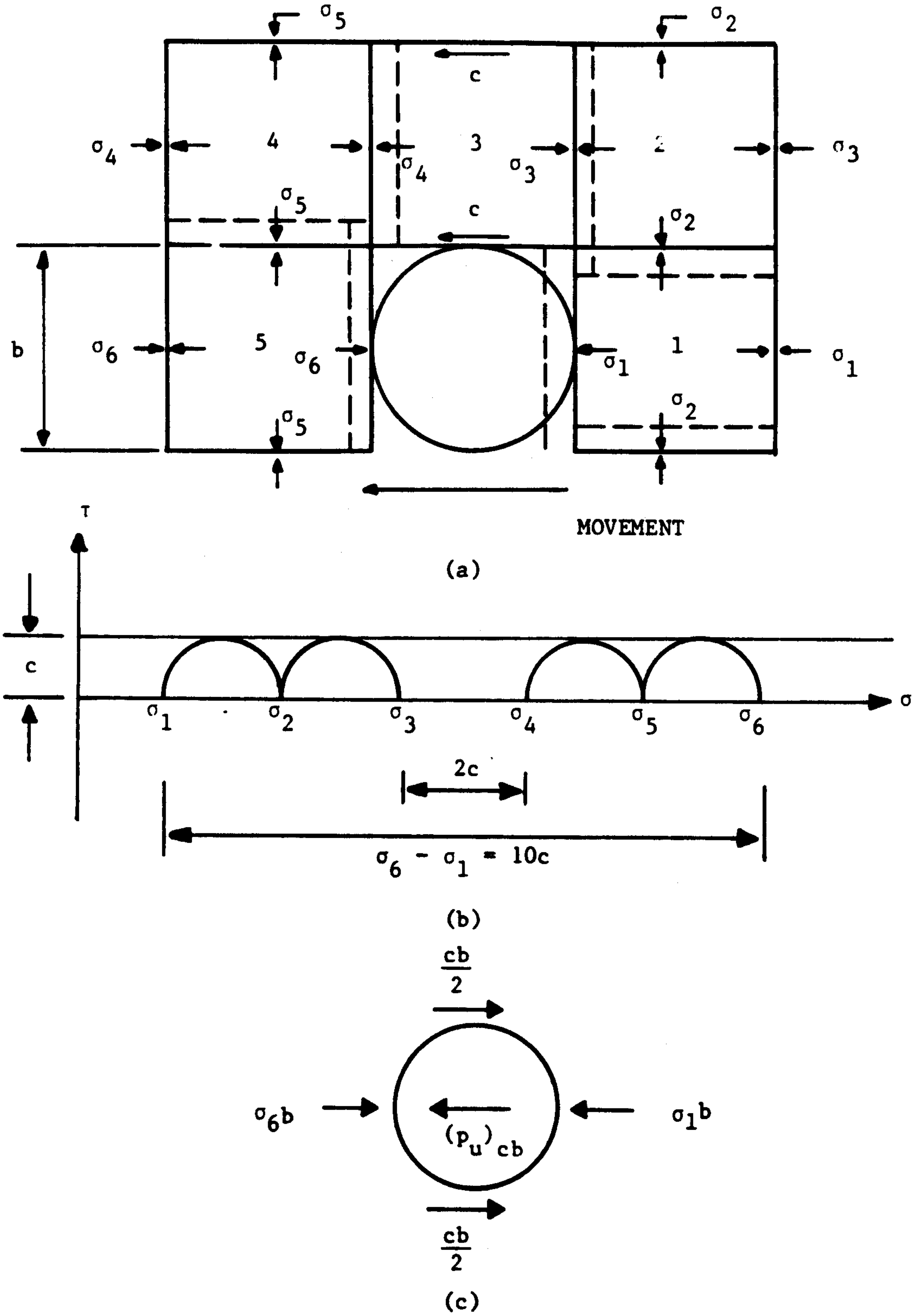


Figure 3.2. Assumed lateral flow-around type of failure for clay: (a) section through pile (b) Mohr-Coulomb diagram (c) forces acting on Pile 4.5.

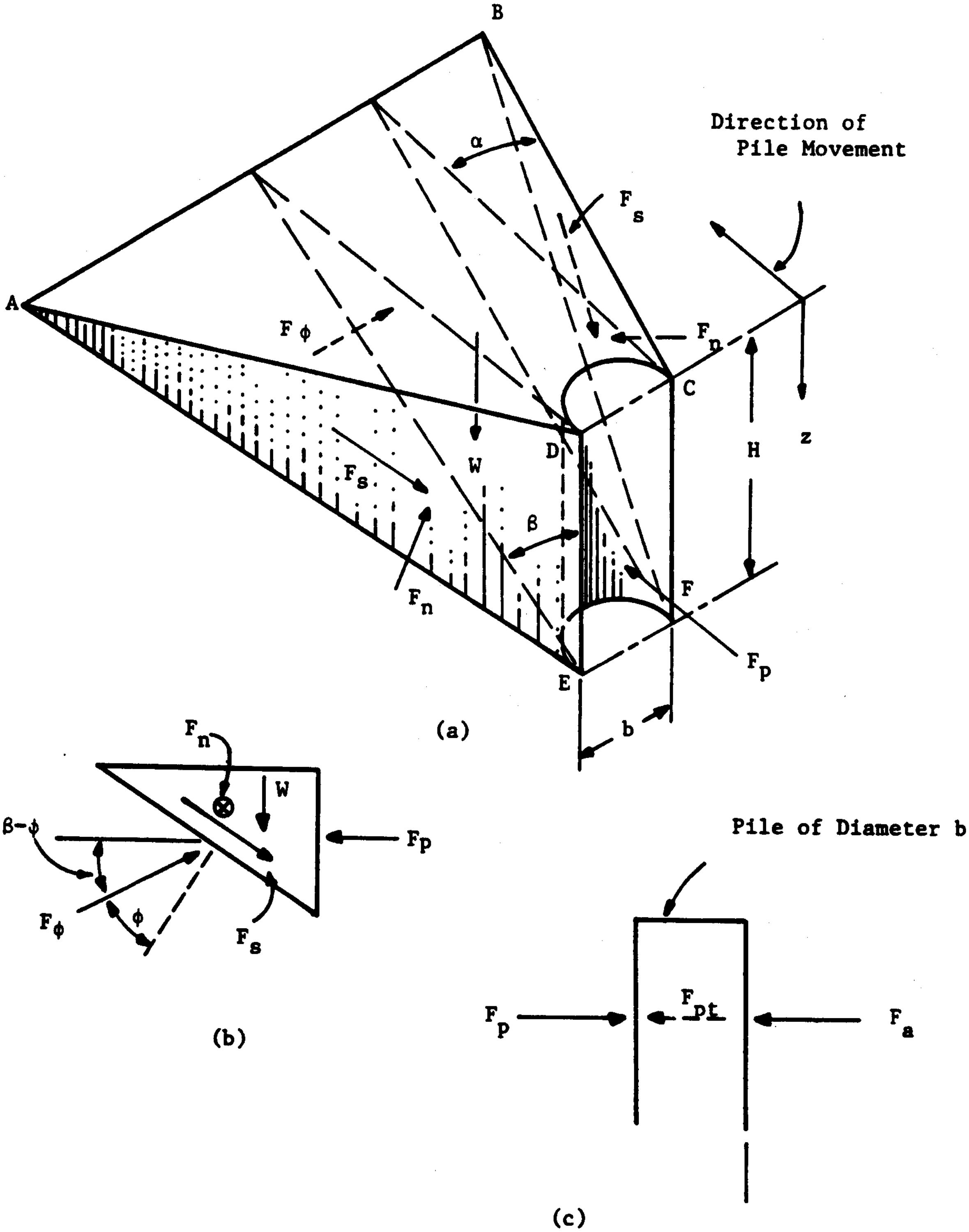


Figure 3.3. Assumed passive wedge-type failure:
(a) general shape of wedge (b) forces of wedge (c) forces on pile
(after Reese, et al, 1974).

be acting on the face of the pile. The equation for F_{pt} is as follows:

$$F_{pt} = \gamma H^2 \left[\frac{K_o H \tan\phi \sin\beta}{3 \tan(\beta-\phi) \cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}\left(\frac{b}{2} + \frac{H}{3}\tan\beta \tan\alpha\right) + \frac{K_o H \tan\beta}{3}(\tan\phi \sin\beta - \tan\alpha) - \frac{K_a b}{2}\right] \quad (3.4)$$

where

K_o = coefficient of earth pressure at-rest, and

K_a = minimum coefficient of active earth pressure.

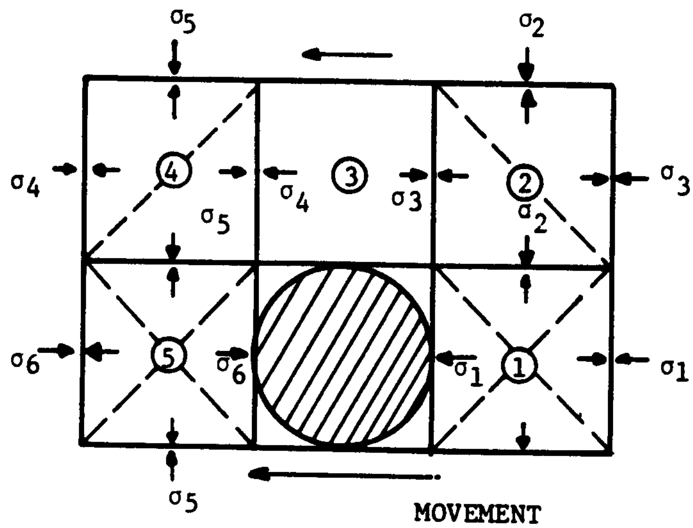
The ultimate soil resistance per unit length of the pile is obtained by differentiating Eq. 3.4.

$$(p_u)_{sa} = \gamma H \left[\frac{K_o H \tan\phi \sin\beta}{\tan(\beta-\phi) \cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}(b + H \tan\beta \tan\alpha) + K_o H \tan\beta (\tan\phi \sin\beta - \tan\alpha) - K_a b\right] \quad (3.5)$$

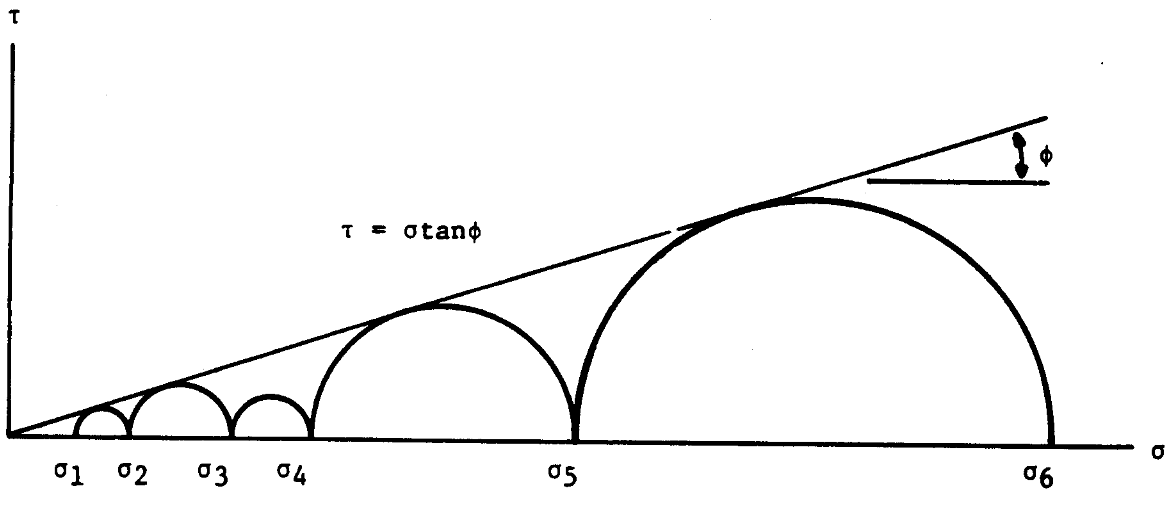
Bowman (1958) suggested values of α from $\phi/3$ to $\phi/2$ for loose sand and up to ϕ for dense sand. The value of β is approximated as follows:

$$\beta = 45 + \frac{\phi}{2} \quad (3.6)$$

The model for computing the ultimate soil resistance at some distance below the ground surface is shown in Fig. 3.4a. The stress σ_1, at the back of the pile must be equal to or larger than the minimum active earth pressure; if not, the soil could fail by



(a)



(b)

Figure 3.4. Assumed mode of soil failure by lateral flow around the pile: (a) section through the pile (b) Mohr-Coulomb diagram representing states of stress of soil flowing around a pile.

slumping. This assumption is based on two-dimensional behavior, of course, and is subject to some uncertainty. However, the assumption should be adequate for the present purpose. Assuming the states of stress shown in Fig. 3.4b, the ultimate soil resistance for horizontal flow around the pile is

$$(p_u)_{sb} = K_a b \gamma H \ (\tan^8 \beta - 1) + K_o b \gamma H \tan\phi \tan^4 \beta. \qquad (3.7)$$

As in the case for clay, Eqs. 3.6 and 3.7 are quite approximate but they serve a useful purpose in indicating the form, if not the magnitude, of the ultimate soil resistance. The two equations can be solved simultaneously to find the approximate depth at which the soil failure changes from the wedge type to the flow-around type.

RECOMMENDATIONS FOR p-y CURVES FOR CLAYS

Three major experimental programs were performed for piles in clays to yield the criteria which follow. In each case the piles were subjected to short-term static loads and to repeated (cyclic) loads. The experimental program is described briefly in the paragraphs that follow; a step-by-step procedure is given for computing the p-y curves, recommendations are given for obtaining the necessary data on soil properties, and example curves are presented.

Response of Soft Clay below the Water Table

Field Experiments

Matlock (1970) performed lateral load tests employing a steel pipe pile that was 12.75 inches in diameter and 42 ft long. It was driven into clays near Lake Austin that had a shear strength of about 800 lb/ft^2. The pile was recovered, taken to Sabine Pass, Texas, and driven into clay with a shear strength that averaged about 300 lb/ft^2 in the significant upper zone.

Recommendations for Computing p-y Curves

The following procedure is for short-term static loading and is illustrated by Fig. 3.5a.

1. Obtain the best possible estimate of the variation of undrained shear strength c and submerged unit weight γ' with depth. Also obtain the values of ε_{50}, the strain corresponding to one-half the maximum principal stress difference. If no stress-strain curves are available, typical values of ε_{50} are given in the following table.

TABLE 3.1. REPRESENTATIVE VALUES OF ε_{50}

Consistency of Clay	ε_{50}
Soft	0.020
Medium	0.010
Stiff	0.005

2. Compute the ultimate soil resistance per unit length of pile, using the smaller of the values given by the equations below.

$$p_u = \left[3 + \frac{\gamma'}{c}x + \frac{J}{b}x\right]cb \qquad (3.8)$$

$$p_u = 9cb \qquad (3.9)$$

where

γ' = average effective unit weight from ground surface to p-y curve,

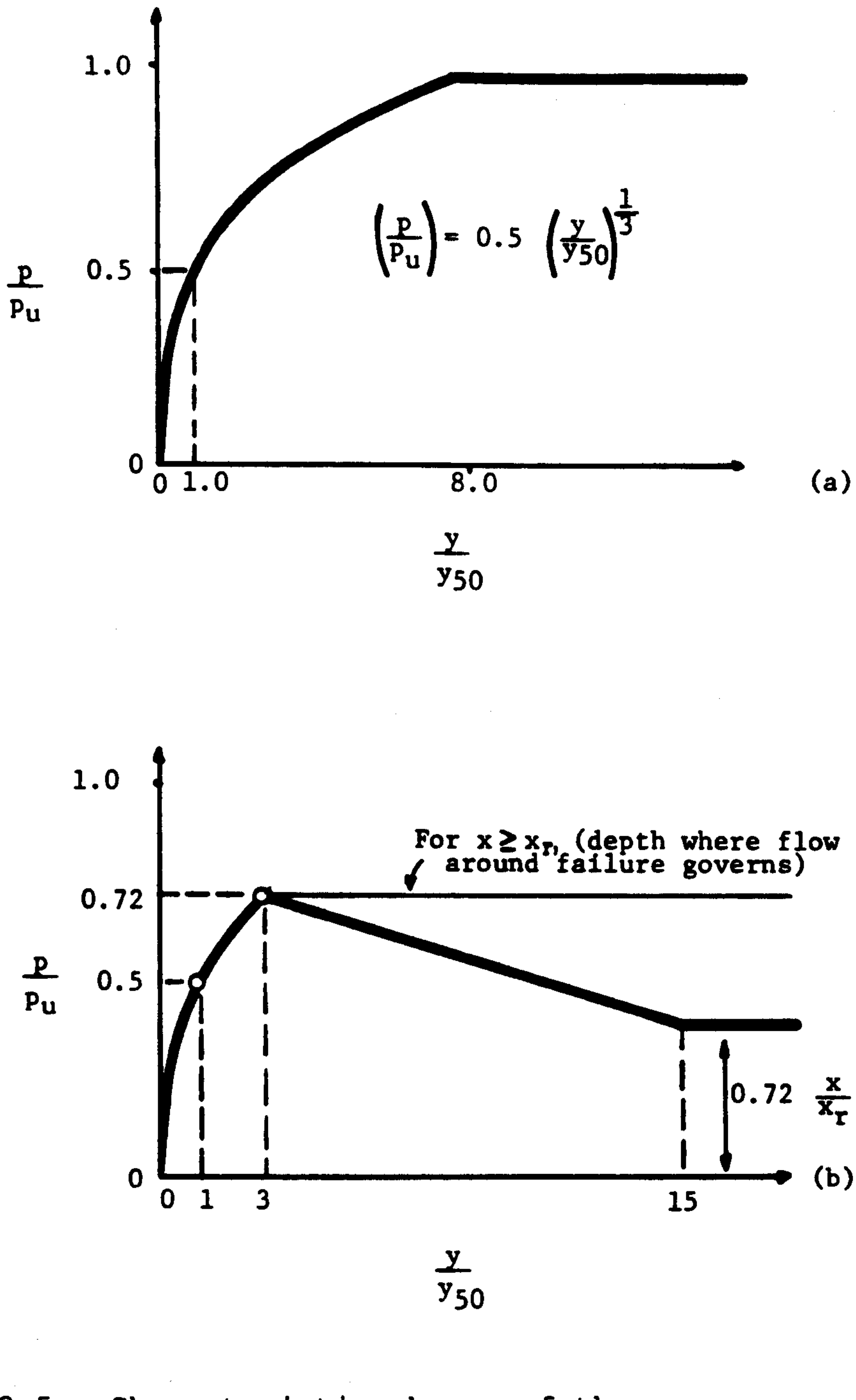


Figure 3.5. Characteristic shapes of the p-y curves for soft clay below water surface: (a) static loading (b) cyclic loading (after Matlock, 1970).

x = depth from ground surface to p-y curve,

c = shear strength at depth x, and

b = width of pile.

Matlock (1970) stated that the value of J was determined experimentally to be 0.5 for a soft clay and about 0.25 for a medium clay. A value of 0.5 is frequently used for J. The value of p_u is computed at each depth where a p-y curve is desired, based on shear strength at that depth.

3. Compute the deflection, y_{50}, at one-half the ultimate soil resistance from the following equation:

$$y_{50} = 2.5\ \varepsilon_{50} b. \tag{3.10}$$

4. Points describing the p-y curve are now computed from the following relationship.

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{\frac{1}{3}} \tag{3.11}$$

The value of p remains constant beyond $y = 8y_{50}$.

The following procedure is for cyclic loading and is illustrated in Fig. 3.5b.

1. Construct the p-y curve in the same manner as for short-term static loading for values of p less than $0.72p_u$.

2. Solve Eqs. 3.8 and 3.9 simultaneously to find the depth, x_r, where the transition occurs. If the unit weight and shear strength are constant in the upper zone, then

$$x_r = \frac{6cb}{(\gamma b + Jc)}. \quad (3.12)$$

If the unit weight and shear strength vary with depth, the value of x_r should be computed with the soil properties at the depth where the p-y curve is desired.

3. If the depth to the p-y curve is greater than or equal to x_r, then p is equal to $0.72p_u$ for all values of y greater than $3y_{50}$.

4 If the depth to the p-y curve is less than x_r, then the value of p decreases from $0.72p_u$ at $y = 3y_{50}$ to the value given by the following expression at $y = 15y_{50}$.

$$p = 0.72p_u \left(\frac{x}{x_r}\right) \quad (3.13)$$

The value of p remains constant beyond $y = 15y_{50}$.

Recommended Soil Tests

For determining the various shear strengths of the soil required in the p-y construction, Matlock (1970) recommended the following tests in order of preference:

1. in-situ vane-shear tests with parallel sampling for soil identification,

2. unconsolidated-undrained triaxial compression tests having a confining stress equal to the overburden pressure with c being defined as half the total maximum principal-stress difference,

3. miniature vane tests of samples in tubes, and

4. unconfined compression tests.

Tests must also be performed to determine the unit weight of the soil.

Response of Stiff Clay below the Water Surface

Field Experiments

Reese, Cox, and Koop (1975) performed lateral-load tests employing steel-pipe piles that were 24 inches in diameter and 50 ft long. The piles were driven into stiff clay at a site near Manor, Texas. The clay had an undrained shear strength ranging from about 1 ton/ft^2 at the ground surface to about 3 ton/ft^2 at a depth of 12 feet.

Recommendations for Computing p-y Curves

The following procedure is for short-term static loading and is illustrated by Fig. 3.6.

1. Obtain values for undrained soil shear strength c, soil submerged unit weight γ', and pile diameter b.

2. Compute the average undrained soil shear strength c_a over the depth x.

3. Compute the ultimate soil resistance per unit length of pile, using the smaller of the values given by the equations below:

$$p_{ct} = 2c_a b + \gamma' bx + 2.83\, c_a x, \text{ and} \tag{3.14}$$

$$p_{cd} = 11cb. \tag{3.15}$$

4. Choose the appropriate value of A_s from Fig. 3.7 for the particular nondimensional depth.

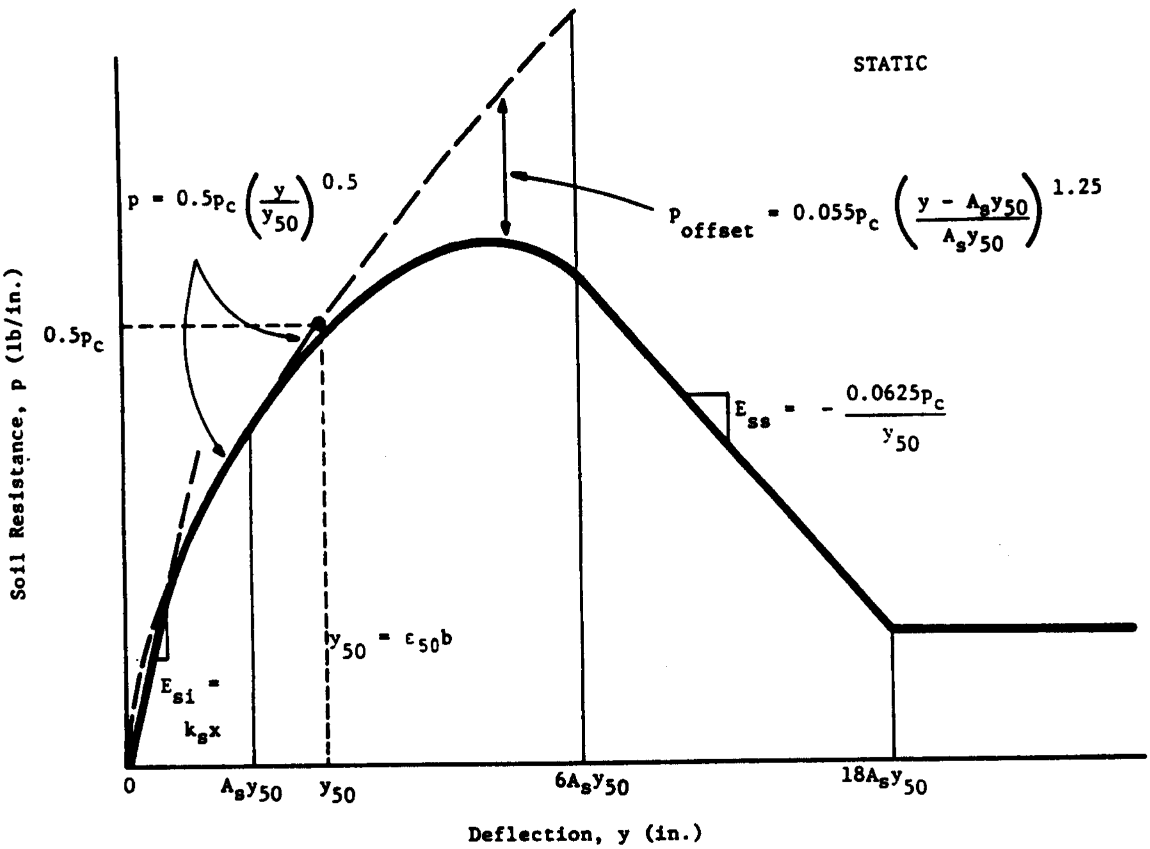


Figure 3.6. Characteristic shape of p-y curve for static loading in stiff clay below the water surface (after Reese, et al, 1975).

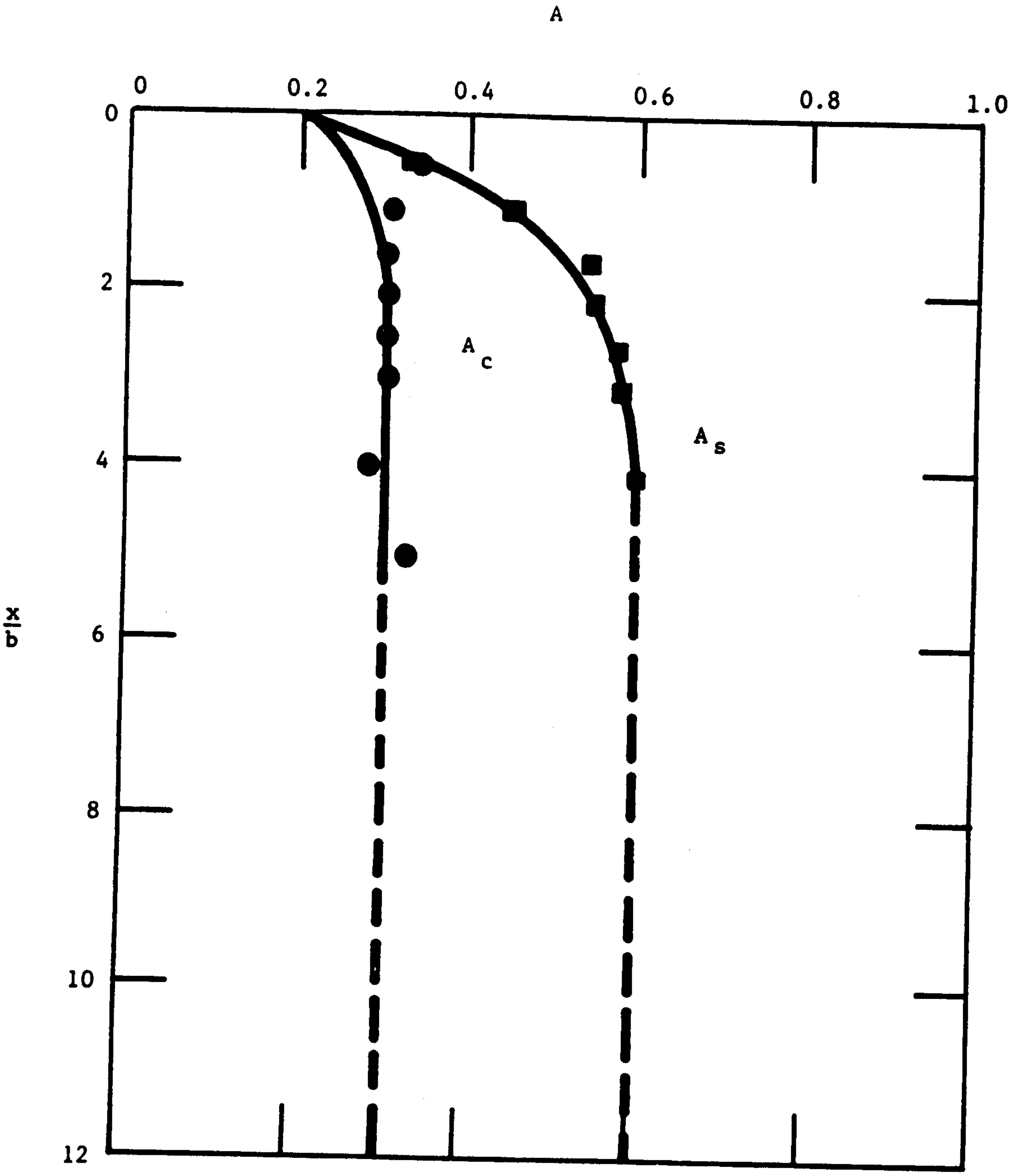


Figure 3.7. Values of constants A_s and A_c (after Reese, et al, 1975).

5. Establish the initial straight-line portion of the p-y curve,

$$p = (kx)y \quad (3.16)$$

Use the appropriate value of k_s or k_c from Table 3.2 for k.

TABLE 3.2. REPRESENTATIVE VALUES OF k FOR STIFF CLAYS

	Average Undrained Shear Strength* ton/ft^2		
	0.5-1	1-2	2.4
k_s (Static) lb/in^3	500	1000	2000
k_c (Cyclic) lb/in^3	200	400	800

*The average shear strength should be computed from the shear strength of the soil to a depth of 5 pile diameters. It should be defined as half the total maximum principal stress difference in an unconsolidated undrained triaxial test.

6. Compute the following:

$$y_{50} = \varepsilon_{50} b \quad (3.17)$$

Use an appropriate value of ε_{50} from results of laboratory tests or, in the absence of laboratory tests, from Table 3.3.

TABLE 3.3. REPRESENTATIVE VALUES OF ε_{50} FOR STIFF CLAYS

	Average Undrained Shear Strength ton/ft^2		
	0.5-1	1-2	2.4
ε_{50} (in/in)	0.007	0.005	0.004

7. Establish the first parabolic portion of the p-y curve, using the following equation and obtaining p_c from Eqs. 3.14 or 3.15.

$$p = 0.5p_c \left(\frac{y}{y_{50}}\right)^{0.5} \quad (3.18)$$

Equation 3.18 should define the portion of the p-y curve from the point of the intersection with Eq. 3.16 to a point where y is equal to $A_s y_{50}$ (see note in Step 10).

8. Establish the second parabolic portion of the p-y curve,

$$p = 0.5p_c \left(\frac{y}{y_{50}}\right)^{0.5} - 0.055p_c \left(\frac{y - A_s y_{50}}{A_s y_{50}}\right)^{1.25}. \quad (3.19)$$

Equation 3.19 should define the portion of the p-y curve from the point where y is equal to $A_s y_{50}$ to a point where y is equal to $6A_s y_{50}$ (see note in Step 10).

9. Establish the next straight-line portion of the p-y curve,

$$p = 0.5p_c(6A_s)^{0.5} - 0.411p_c - \frac{0.0625}{y_{50}} p_c (y - 6A_s y_{50}). \quad (3.20)$$

Equation 3.20 should define the portion of the p-y curve from the point where y is equal to $6A_s y_{50}$ to a point where y is equal to $18A_s y_{50}$ (see note in Step 10).

10. Establish the final straight-line portion of the p-y curve,

$$p = 0.5p_c(6A_s)^{0.5} - 0.411p_c - 0.75p_c A_s, \text{ or} \quad (3.21)$$

$$p = p_c \left(1.225\sqrt{A_s} - 0.75A_s - 0.411\right). \quad (3.22)$$

Equation 3.22 should define the portion of the p-y curve from the point where y is equal to $18A_s y_{50}$ and for all larger values of y (see following note).

Note: The step-by-step procedure is outlined, and Fig. 3.6 is drawn, as if there is an intersection between Eqs. 3.16 and 3.18. However, there may be no intersection of Eq. 3.16 with any of the other equations defining the p-y curve. Equation 3.16 defines the p-y curve until it intersects with one of the other equations or, if no intersection occurs, Eq. 3.16 defines the complete p-y curve.

The following procedure is for cyclic loading and is illustrated in Fig. 3.8.

1. Steps 1, 2, 3, 5, and 6 are the same as for the static case.

4. Choose the appropriate value of A_c from Fig. 3.7 for the particular nondimensional depth.

   Compute the following:

$$y_p = 4.1\ A_c y_{50}. \tag{3.23}$$

7. Establish the parabolic portion of the p-y curve,

$$p = A_c p_c \left[1 - \left| \frac{y - 0.45\ y_p}{0.45\ y_p} \right|^{2.5} \right]. \tag{3.24}$$

   Equation 3.24 should define the portion of the p-y curve from the point of the intersection with Eq. 3.16 to where y is equal to $0.6y_p$ (see note in step 9).

8. Establish the next straight-line portion of the p-y curve,

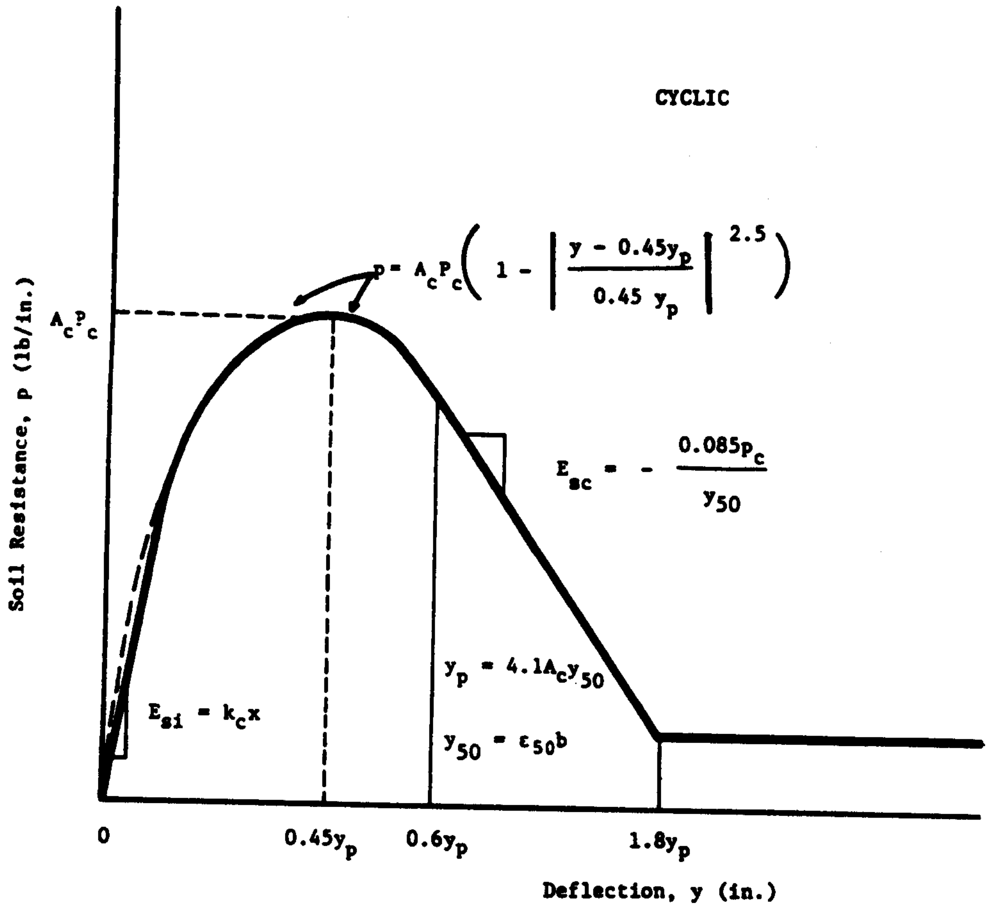


Figure 3.8. Characteristic shape of p-y curve for cyclic loading in stiff clay below water surface (after Reese, et al, 1975).

$$p = 0.936\ A_c p_c - \frac{0.085}{y_{50}}\ p_c\ \left(y - 0.6\ y_p\right). \qquad (3.25)$$

Equation 3.25 should define the portion of the p-y curve from the point where y is equal to $0.6y_p$ to the point where y is equal to $1.8y_p$ (see note in step 9).

9. Establish the final straight-line portion of the p-y curve,

$$p = 0.936\ A_c p_c - \frac{0.102}{y_{50}}\ p_c y_p. \qquad (3.26)$$

Equation 3.26 should define the portion of the p-y curve from the point where y is equal to $1.8y_p$ and for all larger values of y (see following note).

Note: The step-by-step procedure is outlined, and Fig. 3.8 is drawn, as if there is an intersection between Eq. 3.16 and 3.24. However, there may be no intersection of those two equations and there may be no intersection of Eq. 3.16 with any of the other equations defining the p-y curve. If there is no intersection, the equation should be employed that gives the smallest value of p for any value of y.

Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining pressures conforming to in-situ pressures are recommended for determining the shear strength of the soil. The value of ε_{50} should be taken as the strain during the test corresponding to the stress equal to half the maximum total principal stress difference. The shear strength, c, should be interpreted as one-half of the maximum total-stress difference. Values obtained from the triaxial tests might be somewhat conservative but would represent more realistic strength values than other tests. The unit weight of the soil must be determined.

Response of Stiff Clay above the Water Table

Field Experiments

A lateral load test was performed at a site in Houston where the foundation was a drilled shaft, 36 ins in diameter. A 10-inch diameter pipe, instrumented at intervals along its length with electrical-resistance-strain gages, was positioned along the axis of the shaft before concrete was placed. The embedded length of the shaft was 42 feet. The average undrained shear strength of the clay in the upper 20 ft was approximately 2,200 lbs/ft^2. The experiments and their interpretation are discussed in detail by Welch and Reese (1972) and Reese and Welch (1975).

Recommendations for Computing p-y Curves

The following procedure is for short-term static loading and is illustrated in Fig. 3.9.

1. Obtain values for undrained shear strength c, soil unit weight γ, and pile diameter b. Also obtain the values of ε_{50} from stress-strain curves. If no stress-strain curves are available, use a value from ε_{50} of 0.010 or 0.005 as given in Table 3.1, the larger value being more conservative.

2. Compute the ultimate soil resistance per unit length of shaft, p_u, using the smaller of the values given by Eqs. 3.8 and 3.9. (In the use of Eq. 3.8 the shear strength is taken as the average from the ground surface to the depth being considered and J is taken as 0.5. The unit weight of the soil should reflect the position of the water table.)

3. Compute the deflection, y_{50}, at one-half the ultimate soil resistance from Eq. 3.10.

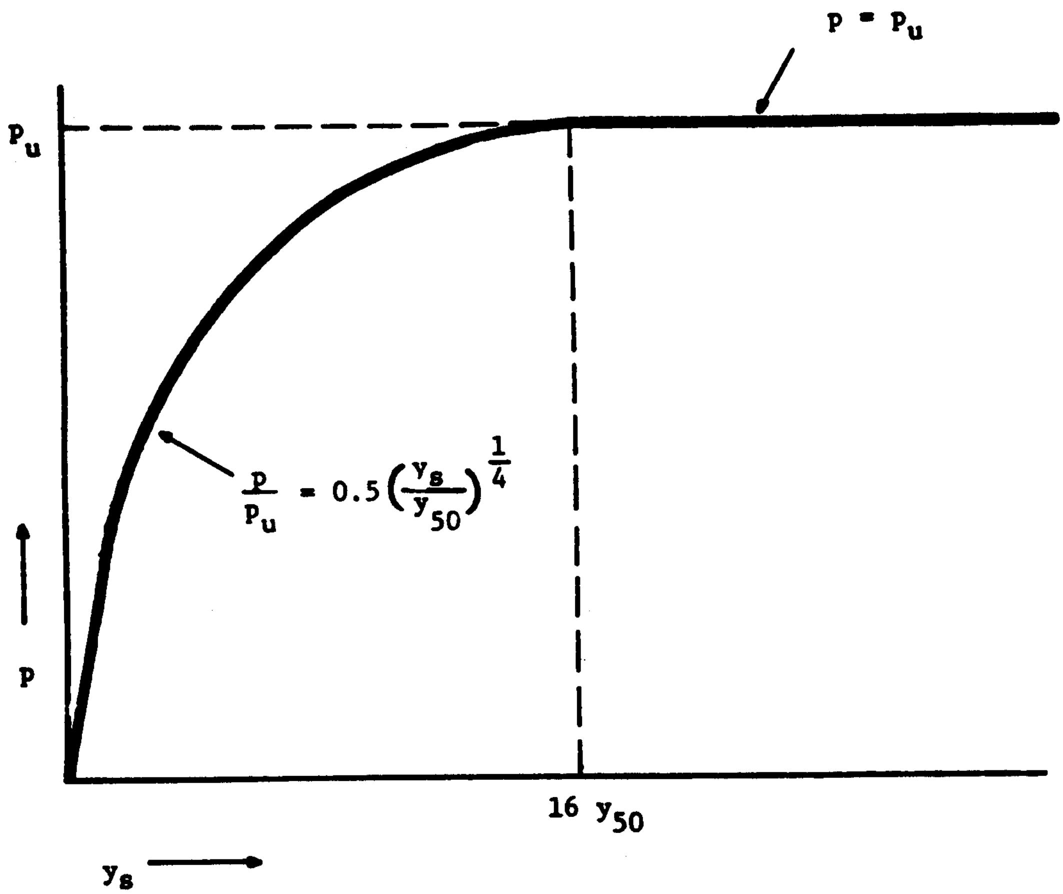


Figure 3.9. Characteristic shape of p-y curve for static loading in stiff clay above water surface (after Welch and Reese, 1972).

4. Points describing the p-y curve may be computed from the relationship below.

$$\frac{p}{p_u} = 0.5 \left(\frac{y}{y_{50}}\right)^{\frac{1}{4}} \qquad (3.27)$$

5. Beyond $y = 16y_{50}$, p is equal to pu for all values of y.

The following procedure is for cyclic loading and is illustrated in Fig. 3.10.

1. Determine the p-y curve for short-term static loading by the procedure previously given.

2. Determine the number of times the design lateral load will be applied to the pile.

3. For several values of p/p_u obtain the value of C, the parameter describing the effect of repeated loading on deformation, from a relationship developed by laboratory tests, (Welch and Reese, 1972), or in the absence of tests, from the following equation.

$$C = 9.6 \left(\frac{p}{p_u}\right)^4 \qquad (3.28)$$

4. At the value of p corresponding to the values of p/p_u selected in step 3, compute new values of y for cyclic loading from the following equation.

$$y_c = y_s + y_{50} \cdot C \cdot \log N \qquad (3.29)$$

where

y_c = deflection under N-cycles of load,

y_s = deflection under short-term static load,

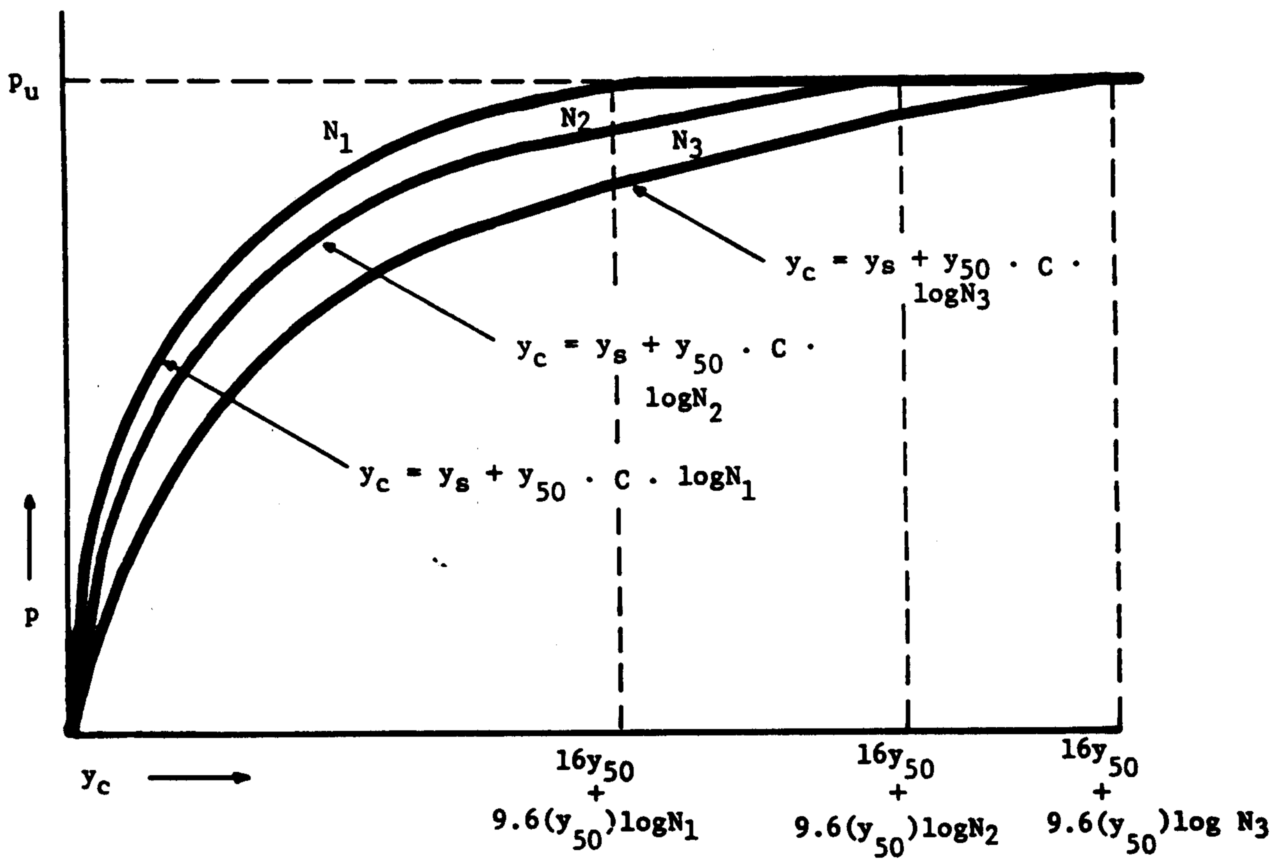


Figure 3.10. Characteristic shape of p-y curve for cyclic loading in stiff clay above water surface (after Welch and Reese, 1972).

y_{50} = deflection under short-term static load at one-half the ultimate resistance, and

N = number of cycles of load application.

5. The p-y curve defines the soil response after N-cycles of load.

Recommended Soil Tests

Triaxial compression tests of the unconsolidated-undrained type with confining stresses equal to the overburden pressures at the elevations from which the samples were taken are recommended to determine the shear strength. The value of ε_{50} should be taken as the strain during the test corresponding to the stress equal to half the maximum total principal stress difference. The undrained shear strength, c, should be defined as one-half the maximum total-principal-stress difference. The unit weight of the soil must also be determined.

RECOMMENDATIONS FOR p-y CURVES FOR SAND

As shown below, a major experimental program was conducted on the behavior of laterally loaded piles in sand below the water table. The results can be extended to sand above the water table.

Response of Sand below the Water Table

Field Experiments

An extensive series of tests was performed at a site on Mustang Island, near Corpus Christi (Cox, Reese, and Grubbs, 1974). Two steel pipe piles, 24 ins in diameter, were driven into sand in a manner to simulate the driving of an open-ended pipe,

pipe, and were subjected to lateral loading. The embedded length of the piles was 69 feet. One of the piles was subjected to short-term loading and the other to repeated loading.

The soil at the site was a uniformly graded, fine sand with an angle of internal friction of 39 degrees. The submerged unit weight was 66 lb/ft^3. The water surface was maintained a few inches above the mudline throughout the test program.

Recommendations for Computing p-y Curves

The following procedure is for short-term static loading and for cyclic loading and is illustrated in Fig. 3.11 (Reese, Cox, and Koop, 1974).

1. Obtain values for the angle of internal friction ϕ, the soil unit weight γ, and pile diameter b.

2. Make the following preliminary computations.

$$\alpha = \frac{\phi}{2};\ \beta = 45 + \frac{\phi}{2};\ K_o = 0.4;\ \text{and}\ K_a = \tan^2\left(45 - \frac{\phi}{2}\right) \tag{3.30}$$

3. Compute the ultimate soil resistance per unit length of pile using the smaller of the values given by the equations below.

$$p_{st} = \gamma x \left[\frac{K_o x \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}(b + x\tan\beta\tan\alpha) + K_o x \tan\beta(\tan\phi\sin\beta - \tan\alpha) - K_a b\right]. \tag{3.31}$$

$$p_{sd} = K_a b\gamma x(\tan^8\beta - 1) + K_o b\gamma x\tan\phi\tan^4\beta. \tag{3.32}$$

For the sand below the water table, the submerged unit weight γ' should be used.

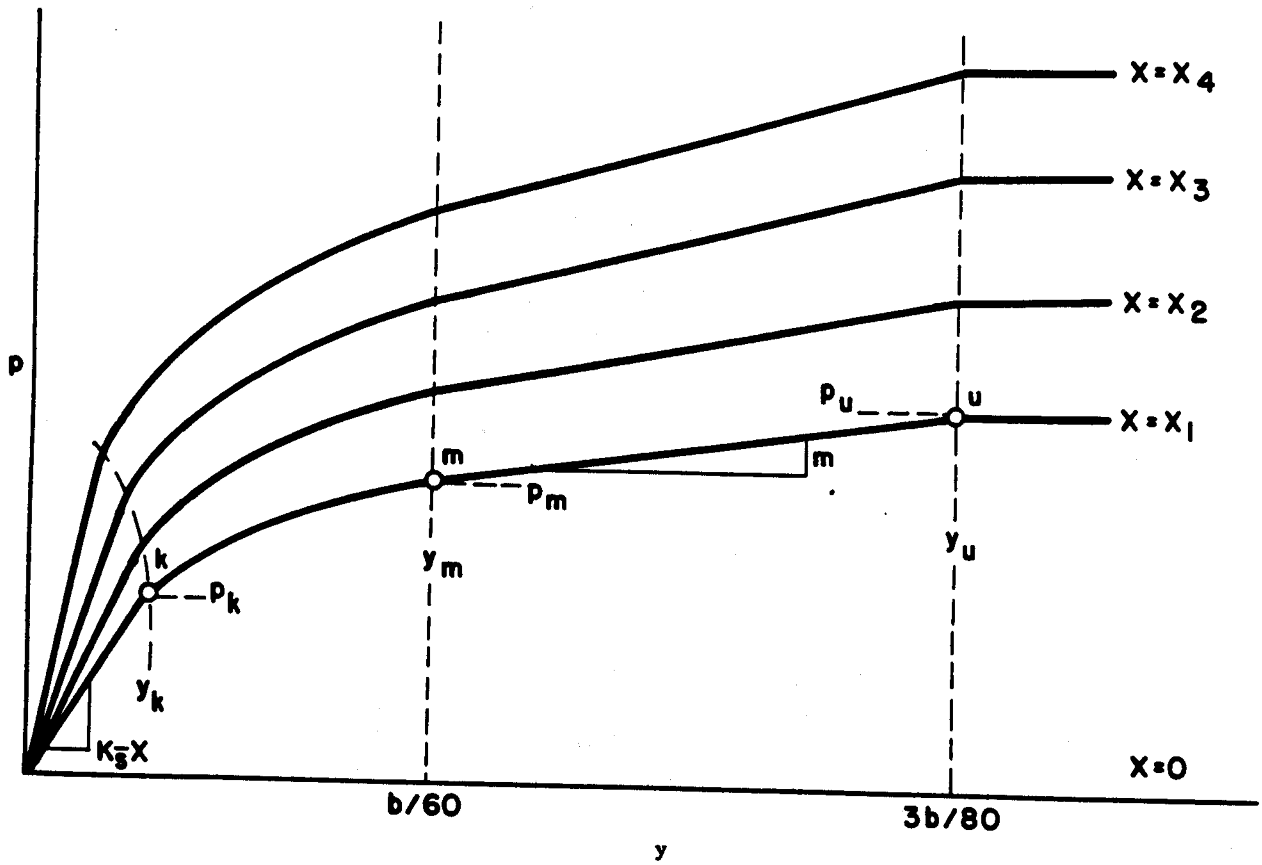


Figure 3.11. Characteristic shape of a family of p-y curves for static and cyclic loading in sand (after Reese, et al, 1974).

4. In making the computations in Step 3, find the depth x_t at which there is an intersection at Eqs. 3.31 and 3.32. Above this depth use Eq. 3.31. Below this depth use Eq. 3.32.

5. Select a depth at which a p-y curve is desired.

6. Establish y_u as 3b/80. Compute p_u by the following equation:

$$p_u = \bar{A}_s p_s \text{ or } p_u = \bar{A}_c p_s. \qquad (3.33)$$

   Use the appropriate value of $\bar{A}_s$ or $\bar{A}_c$ from Fig. 3.12 for the particular nondimensional depth, and for either the static or cyclic case. Use the appropriate equation for p_s, Eq. 3.31 or Eq. 3.32, by referring to the computation in step 4.

7. Establish y_m as b/60. Compute p_m by the following equation:

$$p_m = B_s p_s \text{ or } p_m = B_c p_s. \qquad (3.34)$$

   Use the appropriate value of B_s or B_c from Fig. 3.13 for the particular nondimensional depth, and for either the static or cyclic case. Use the appropriate equation for p_s. The two straight-line portions of the p-y curve, beyond the point where y is equal to b/60, can now be established.

8. Establish the initial straight-line portion of the p-y curve,

$$p = (kx)y. \qquad (3.35)$$

   Use the appropriate value of k from Tables 3.4 or 3.5.

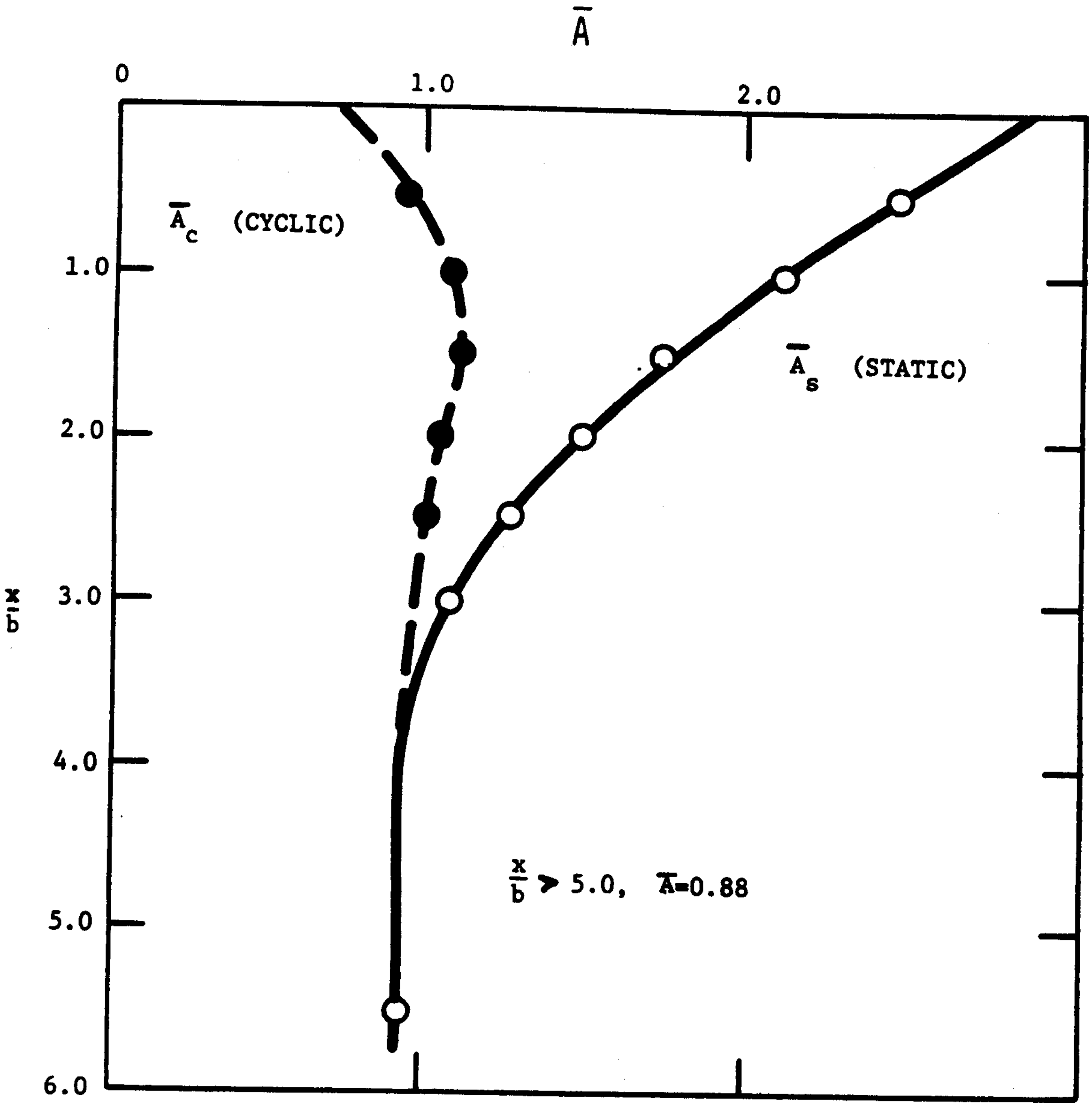


Figure 3.12. Values of coefficients $\bar{A}_c$ and $\bar{A}_s$ (after Reese, et al, 1974).

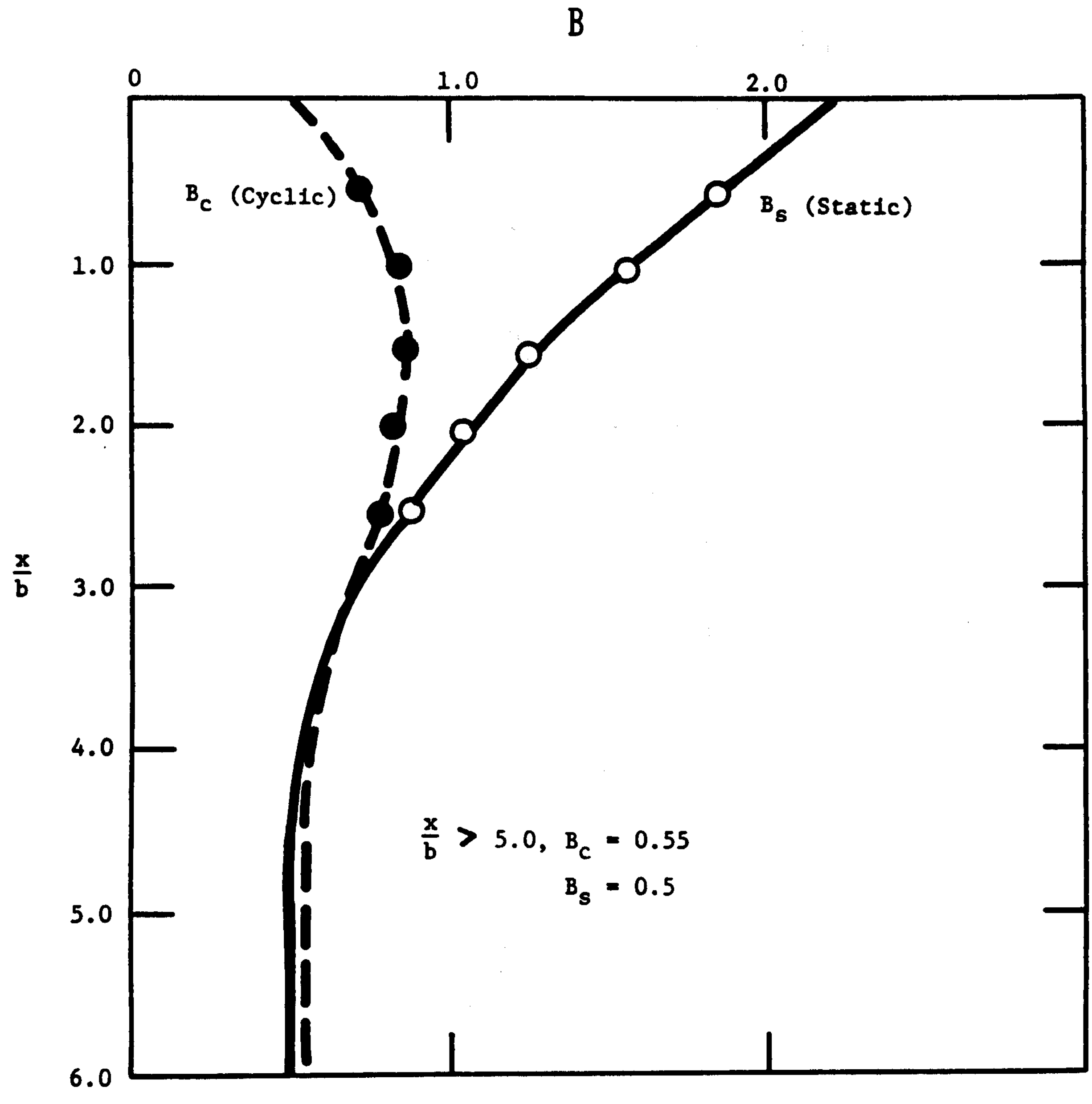


Figure 3.13. Values of coefficient B for soil resistance versus depth (after Reese, et al, 1974).

TABLE 3.4. REPRESENTATIVE VALUES OF k FOR SUBMERGED SAND (Static and Cyclic Loading)

Relative Density	Loose	Medium	Dense
Recommended k (lb/in^3)	20	60	125

TABLE 3.5. REPRESENTATIVE VALUES OF k FOR SAND ABOVE WATER TABLE (Static and Cyclic Loading)

Relative Density	Loose	Medium	Dense
Recommended k (lb/in^3)	25	90	225

9. Establish the parabolic section of the p-y curve,

$$p = \overline{C}\, y^{1/n}. \qquad (3.36)$$

Fit the parabola between points k and m as follows:

a. Get the slope of line between points m and u by,

$$m = \frac{p_u - p_m}{y_u - y_m}. \qquad (3.37)$$

b. Obtain the power of the parabolic section by,

$$n = \frac{p_m}{m y_m}. \qquad (3.38)$$

c. Obtain the coefficient $\overline{C}$ as follows:

$$\overline{C} = \frac{p_m}{y_m^{1/n}}. \qquad (3.39)$$

d. Determine point k as,

$$y_k = \left(\frac{\bar{C}}{kx}\right)^{n/n-1} . \qquad (3.40)$$

e. Compute appropriate number of points on the parabola by using Eq. 3.36.

Note: The step-by-step procedure is outlined, and Fig. 3.11 is drawn, as if there is an intersection between the initial straight-line portion of the p-y curve and the parabolic portion of the curve at point k. However, in some instances there may be no intersection with the parabola. Equation 3.35 defines the p-y curve until there is an intersection with another branch of the p-y curve or if no intersection occurs, Eq. 3.35 defines the complete p-y curve. This completes the development of the p-y curve for the desired depth. Any number of curves can be developed by repeating the above steps for each desired depth.

Recommended Soil Tests

Triaxial compression tests are recommended for obtaining the angle of internal friction of the sand. Confining pressures should be used which are close or equal to those at the depths being considered in the analysis. Tests must be performed to determine the unit weight of the sand.

Response of Sand above the Water Table

The procedure in the previous section can be used for sand above the water table if appropriate adjustments are made in the unit weight and angle of internal friction of the sand. Some small-scale experiments were performed by Parker and Reese (1971) and recommendations for p-y curves for dry sand were developed from the experiments. The results from the Parker and Reese experiments should be useful as a check of solutions made using results from the test program using full-scale piles.

RECOMMENDATIONS FOR p-y CURVES FOR VUGGY LIMESTONE

Field Experiments

Very little information is available on the behavior of piles that have been installed in rock. Some other type of foundation would normally be used. However, a study was made of the behavior of an instrumented drilled shaft that was installed in vuggy limestone in the Florida Keys (Reese and Nyman, 1978). The test was performed for the purpose of gaining information for the design of foundations for highway bridges.

Difficulty was encountered in obtaining properties of the intact rock. Cores broke during excavation and penetrometer tests were misleading (because of the vugs) or could not be run. Tests were made on two cores from the site. The small discontinuities in the outside surface of the specimens were covered with a thin layer of gypsum cement in an effort to minimize stress concentrations. The ends of the specimens were cut with a rock saw and lapped flat and parallel. The specimens were 5.88 ins in diameter and with heights of 11.88 ins for Specimen 1 and 10.44 ins for Specimen 2. The undrained shear strength of the specimens were taken as one-half the unconfined compressive strength and were 17.4 and 13.6 T/sq ft for Specimens 1 and 2, respectively.

The rock at the site was also investigated by in-situ-grout-plug tests under the direction of Dr. John Schmertmann (1977). A 5.5-inch diameter hole was drilled into the limestone, a high strength steel bar was placed to the bottom of the hole, and a grout plug was cast over the lower end of the bar. The bar was pulled until failure occurred and the grout was examined to see that failure occurred at the interface of the grout and limestone. Tests were performed at three borings and the following results were obtained, in T/sq ft; depth into limestone from 2.5 to 5 ft, 23.8, 13.7, and 12.0; depth into limestone from 8 to 10 ft, 18.2, 21.7, and 26.5; depth into limestone from 18 to 20 ft, 13.7 and 10.7. The average of the eight tests was 16.3 T/sq feet. However, the rock was stronger in the zone where the deflections of the drilled shaft were most significant and a shear strength of 18 T/sq ft was selected for correlation.

The drilled shaft was 48 inches in diameter and penetrated 43.7 ft into the limestone. The overburden of fill was 14 ft thick and was cased. The load was applied about 11.5 ft above the limestone. A maximum horizontal load of 75 tons was applied to the drilled shaft. The maximum deflection at the point of load application was 0.71 in and at the top of the rock (bottom of casing) it was 0.0213 inch. While the curve of load versus deflection was nonlinear, there was no indication of failure of the rock.

Recommendations for Computing p-y Curves

A single p-y curve, shown in Fig. 3.14, was proposed for the design of piles under lateral loading in the Florida Keys. Data are insufficient to indicate a family of curves to reflect any increased resistance of the rock.

As shown in the figure, load tests are recommended if deflection of the rock (and pile) are greater than 0.0004b and brittle fracture is assumed if the lateral stress (force per unit length) against the rock becomes greater than the diameter times the shear strength s_u of the rock.

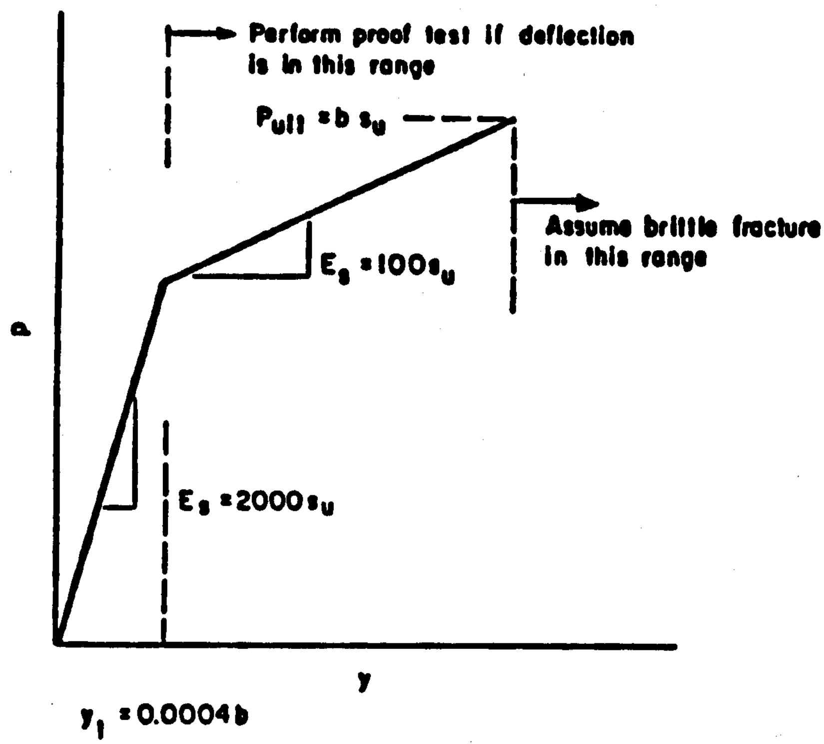


Figure 3.14. Recommended p-y curve for design of drilled shafts in vuggy limestone (after Reese and Nyman, 1978).

The p-y curve shown in Fig. 3.14 should be employed with considerable caution because of the limited amount of experimental data and because of the great variability in rock. The behavior of rock at a site could be very well controlled by joints, cracks, and secondary structure of rock and not by the strength of intact specimens.

RECOMMENDATIONS FOR p-y CURVES FOR LAYERED SOIL

There are numerous cases where the soil near the ground surface is not homogeneous but is layered. If the layers are in the zone where the soil would move up and out as a wedge, some modification is plainly needed in order to compute the ultimate soil resistance p_u, and consequently modifications are needed in the p-y curves.

The problem of the layered soil has been given intensive study by Allen (1985); however, Allen's formulations require the use of several computer codes. Integrating the methods of Allen with the methods shown herein must be delayed until a later date when his research can be put in a readily usable form.

Method of Georgiadis

The proposal of Georgiadis (1983) was selected for the purposes of the computer code that is presented here. The method is based on the determination of the "equivalent" depth of all the layers existing below the upper layer. The p-y curves of the upper layer are determined according to the methods presented herein for homogeneous soils. To compute the p-y curves of the second layer, the equivalent depth H_2 to the top of the second layer has to be determined by summing the ultimate resistances of the upper layer and equating that value to the summation as if the upper layer had been composed of the same material as in the second layer. The values of p_u are computed according to the equations given earlier. Thus, the following two equations are solved simultaneously for H_2:

$$F_1 = \int_0^{H_1} p_{u1} \, dH, \text{ and} \tag{3.41}$$

$$F_1 = \int_0^{H_2} p_{u2} \, dH. \tag{3.42}$$

The equivalent thickness H_2 of the upper layer along with the soil properties of the second layer, are used to compute the p-y curves for the second layer.

The concepts presented above can be used to get the equivalent thickness of two or more dissimilar layers of soil overlying the layer for whom the p-y curves are desired.

Example p-y Curves

The example problem to demonstrate the manner in which the computer program deals with layered soils is shown in Fig. 3.15. As seen in the sketch, a pile with a diameter of 24 ins is embedded in soil consisting of an upper layer of soft clay, overlying a layer of loose sand, which in turn overlays a layer of stiff clay. The water table is at the ground surface and the loading is assumed to be static.

Four p-y curves for the case of layered soil are shown in Fig. 3.16. The curve at a depth of 36 ins falls in the upper zone of soft clay; the curve for the depth of 72 ins falls in the sand

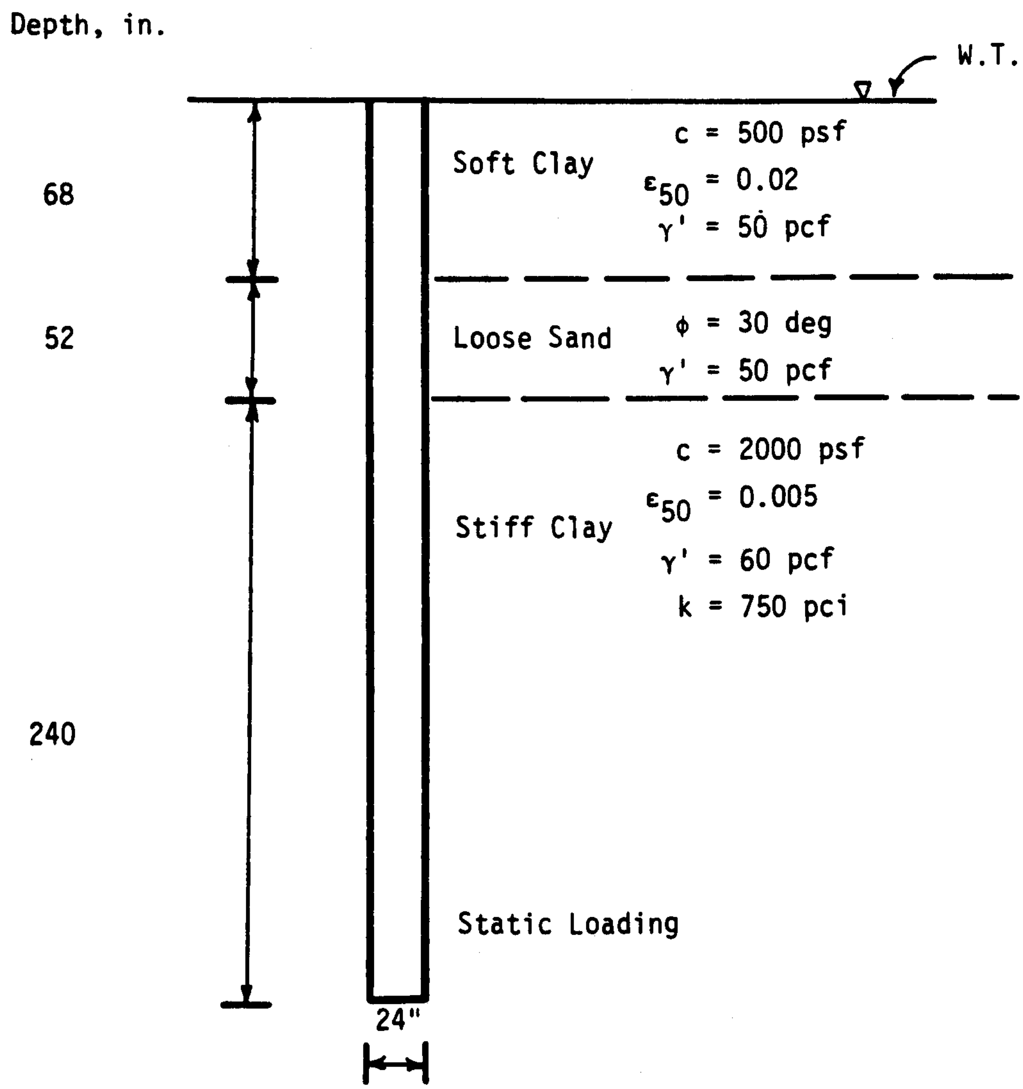


Figure 3.15. Example problem for soil response for layered soils.

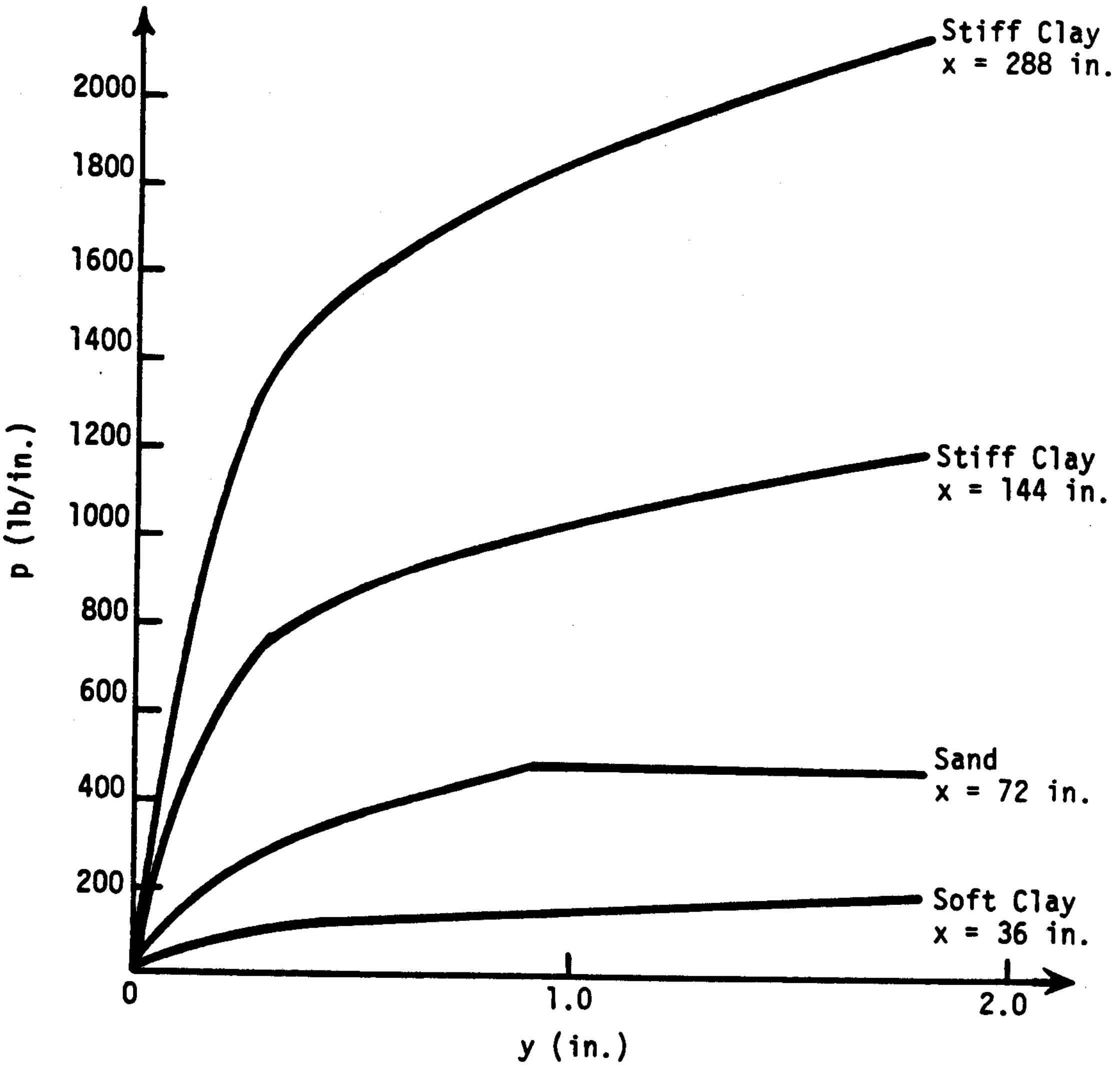


Figure 3.16. Example p-y curves for layered soils.