APPENDIX B

GUIDELINES FOR FOUNDATION DESIGN OF DIRECTLY EMBEDDED OVERHEAD LINE POLES FOR LATERAL LOADS AND MOMENTS

B1 VARIABLES

ϕ	=	soil angle of friction	
γ	=	soil density	(kN/m^2)
c	=	soil cohesion	(kPa)
COV	=	coefficient of variation	
D	=	'effective diameter' of foundation	(m)
H	=	ground line lateral load	(kN)
H_L	=	nominal failure load	(kN)
H_{calc}	=	calculated value using recommended method	(kN)
$H_{max.}$	=	maximum lateral load	(kN)
K_i	=	factor that is function of soil modulus of elasticity and foundation geometry	
K_q, K_c	=	factors that are a function of z/D and ϕ	
L	=	trial embedment depth	(m)
M	=	bending moment at ground line	(kNm)
p	=	ultimate soil pressure	(kPa)
q_z	=	vertical overburden pressure at depth z, $q_z = \gamma z$	(kPa)
z	=	depth below the ground surface	(m)
z_r	=	point of rotation at an unknown depth below the surface	(m)

B2 LOAD DISPLACEMENT CHARACTERISTICS AND FAILURE CRITERION

The load displacement relationship for laterally loaded piles (pole foundations) is highly non-linear with no clearly defined failure load. Figure B1 shows a typical load displacement plot.

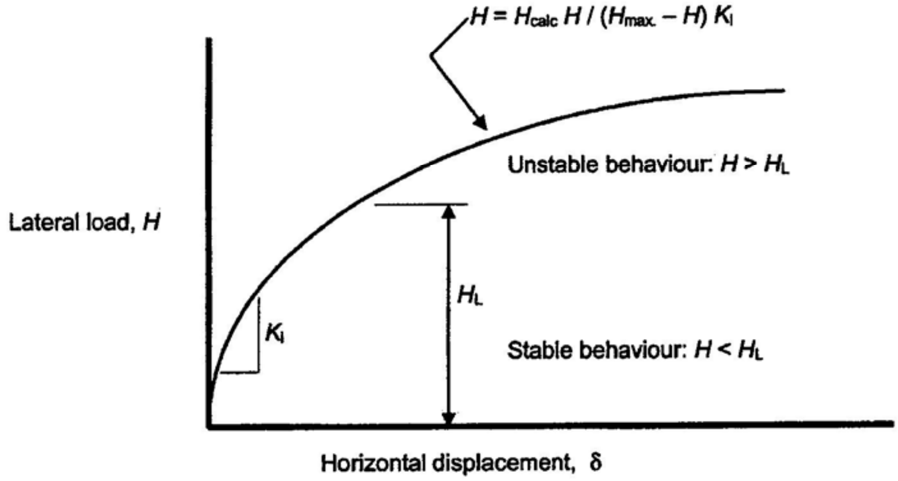

FIGURE B1 TYPICAL LOAD DISPLACEMENT PLOT

The 'failure' load (H_L) predicted by the method presented here represents a threshold load level at which soil failure is initiated. Below this level the soil/pole system demonstrates 'stable' behaviour whereas the system becomes 'unstable' above this level.

The Brinch Hansen method presented here is considered to be appropriate to the dimensional range and characteristics of poles in transmission and distribution line structures. The method is applicable to a wide variety of soil types and provides consistent results. Typically, the correlation between predicted and observed test results has been:

(a) undrained conditions: $H_L = 1.01\ H_{calc}$ with $COV = 0.36$

(b) drained conditions: $H_L = 0.60\ H_{calc}$ with $COV = 0.37$

where

H_L = nominal failure load

H_{calc} = calculated value using recommended method

COV = coefficient of variation

It should be borne in mind that the accuracy of any solution will be limited by the accuracy of the input data. The appropriate component strength factor (Table 3.1) should be applied to H_L.

The Brinch Hansen method does not provide an indication of the pole rotation at the H_L load. This should be calculated separately using methods recommended in AS 2159 or another suitable source. (As a general indication, ground line displacements of 25 to 50 mm may be expected at H_L, though the centre of rotation is dependent on the foundation geometry and soil parameters.) Note that if the load displacement plot is assumed to be hyperbolic and the initial slope and H_{max}. value are known, then values along the curve may be calculated. The initial slope is dependent on the modulus of elasticity for the soil and the foundation geometry.

B3 ANALYTICAL PROCEDURE FOR DETERMINATION OF FAILURE LOAD/MOMENT

The mathematical model of the pole/soil system is shown in Figure B2.

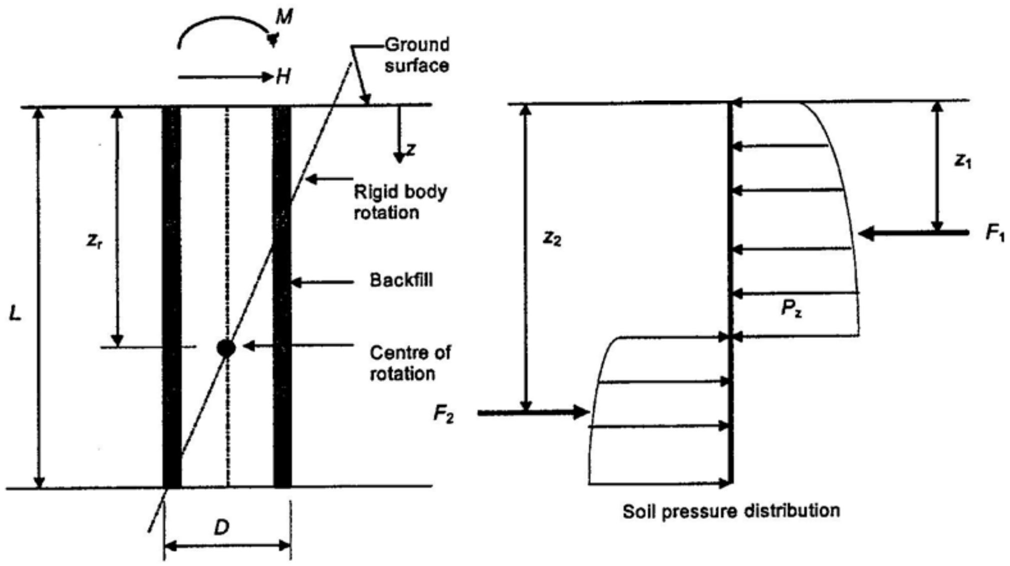

FIGURE B2 MODEL OF THE POLE/SOIL SYSTEM

The system is subjected to a ground line lateral load, H, and bending moment, M. The 'effective diameter', D, can be taken as the average pole diameter below ground for soil backfill situations and the auger diameters for situations where concrete or soil/cement backfill is used.

The pole is assumed to rotate as a rigid body under the applied loads about a point of rotation at an unknown depth, z_r, below the surface. At the point of failure this rotation produces a soil stress distribution as depicted in Figure B2 with the ultimate soil pressure, p, varying with depth below the ground surface, z.

The ultimate lateral soil resistance at any depth, z, below the surface can be expressed as:

$$P_z = q_z K_q + c_u K_c$$

where

q_z = vertical overburden pressure at depth $z = \gamma z$

γ = soil density (see Table B5)

c_u = soil cohesion (see Table B3)

K_q, K_c = factors that are a function of z/D and the soil angle of friction, ϕ (see Table B4)

Values of K_q are given in Table B1, and those of K_c are plotted in Table B2.

The limiting combination of H and M to cause failure may be obtained by considering the equilibrium of horizontal forces and moments, and solving the resulting simultaneous equations for the unknown depth of the centre of rotation, z_r. In general form the equations are:

Horizontal equilibrium

$$H = F_1 - F_2$$

where

$$F_2 = \int_0^{z_r} p_z D dz$$

$$F_2 = \int_{z_r}^{L} p_z D dz$$

Moment equilibrium

$$M = F_2 z_2 - F_1 z_1$$

where

z_1 = distance to resultant load F_1

z_2 = distance to resultant load F_2

It is usually more convenient to solve the resulting equations by trial and error. That is, for a given horizontal load, H, and a trial embedment depth, L, the unknown depth of rotation, z_r, and moment, M, can be determined. The process is repeated by varying L until the required M is obtained.

For non-cohesive soils, e.g. dry sand, the depth of rotation is typically 2/3 of the total depth. For cohesive soils, e.g. clayey sands, the depth of rotation is typically slightly more than half depth. As the eccentricity of load increases z_r converges to either 2/3 or 1/2 of the total depth.

Where a bed log is used the calculated soil forces F_1 and F_2 may be based on the Brinch Hansen method. The forces should be based on soil pressure p_z and the areas of the bed log and the pole foundation.

TABLE B1

EARTH PRESSURE COEFFICIENT FOR OVERBURDEN PRESSURE, K_q

Angle of friction ϕ

z/D	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°
1.0	0	0.50	1.10	1.85	2.81	4.12	5.99	8.85	13.50	21.81
1.5	0	0.52	1.16	1.97	3.02	4.46	6.53	9.67	14.75	23.72
2.0	0	0.53	1.21	2.07	3.21	4.76	7.02	10.44	15.96	25.59
2.5	0	0.55	1.26	2.16	3.37	5.04	7.46	11.17	17.12	27.43
3.0	0	0.56	1.30	2.24	3.51	5.28	7.88	11.86	18.24	29.23
3.5	0	0.57	1.33	2.32	3.64	5.50	8.26	12.50	19.32	31.00
4.0	0	0.58	1.36	2.38	3.75	5.70	8.61	13.12	20.37	32.74
4.5	0	0.59	1.39	2.44	3.86	5.88	8.93	13.70	21.38	34.45
5.0	0	0.60	1.42	2.49	3.95	6.05	9.24	14.25	22.36	36.13
6.0	0	0.62	1.46	2.58	4.11	6.35	9.79	15.27	24.23	39.39
7.0	0	0.63	1.50	2.65	4.25	6.60	10.27	16.20	25.98	42.55
8.0	0	0.64	1.53	2.71	4.37	6.82	10.69	17.05	27.63	45.59
9.0	0	0.65	1.56	2.77	4.47	7.02	11.07	17.82	29.18	48.54
10.0	0	0.66	1.58	2.82	4.56	7.19	11.41	18.53	30.64	51.39
12.0	0	0.68	1.62	2.89	4.71	7.47	12.00	19.79	33.34	56.81
14.0	0	0.69	1.65	2.96	4.82	7.70	12.49	20.88	35.77	61.90
16.0	0	0.70	1.68	3.01	4.92	7.89	12.90	21.82	37.96	66.69
18.0	0	0.71	1.70	3.05	5.00	8.05	13.25	22.65	39.95	71.20
20.0	0	0.72	1.72	3.08	5.07	8.19	13.55	23.38	41.77	75.46

TABLE B2

EARTH PRESSURE COEFFICIENT FOR COHESION, K_C

Angle of friction ϕ

z/D	~0°	5°	10°	15°	20°	25°	30°	35°	40°	45°
1.0	4.8	5.7	6.8	8.2	10.2	12.9	16.9	22.8	31.9	47.2
1.5	5.3	6.4	7.7	9.5	11.9	15.4	20.6	28.4	40.8	61.3
2.0	5.7	6.9	8.4	10.5	13.3	17.4	23.7	33.5	49.1	75.0
2.5	6.0	7.3	9.0	11.2	14.4	19.1	26.4	38.0	56.8	88.1
3.0	6.2	7.6	9.4	11.8	15.3	20.5	28.7	42.0	63.9	100.7
3.5	6.4	7.9	9.8	12.4	16.1	21.7	30.8	45.7	70.6	112.8
4.0	6.6	8.1	10.1	12.8	16.7	22.7	32.6	49.0	76.9	124.5
4.5	6.7	8.3	10.3	13.1	17.3	23.6	34.2	52.1	82.8	135.8
5.0	6.8	8.4	10.5	13.4	17.7	24.4	35.6	54.8	88.4	146.7
6.0	7.0	8.7	10.9	13.9	18.5	25.8	38.0	59.8	98.6	167.4
7.0	7.1	8.8	11.1	14.3	19.1	26.8	40.1	64.0	107.7	186.7
8.0	7.2	9.0	11.3	14.7	19.7	27.7	41.8	67.6	115.9	204.8
9.0	7.3	9.1	11.5	14.9	20.1	28.5	43.2	70.8	123.3	221.8
10.0	7.4	9.2	11.7	15.1	20.4	29.1	44.5	73.6	130.1	237.8
12.0	7.5	9.4	11.9	15.5	21.0	30.1	46.5	78.3	141.9	267.1
14.0	7.6	9.5	12.0	15.7	21.4	30.9	48.1	82.1	151.9	293.3
16.0	7.6	9.6	12.2	15.9	21.7	31.5	49.4	85.3	160.4	316.8
18.0	7.7	9.6	12.3	16.1	22.0	32.0	50.5	87.9	167.8	338.0
20.0	7.7	9.7	12.4	16.2	22.2	32.4	51.3	90.2	174.3	357.3

The over burden pressure and earth pressure coefficients, K_q^z, K_c^z at depth z as given in the table above can be calculated from the formulae below.

NOTE: For more information on these formulas refer to the original Brinch Hansen paper.

$$K_0 = 1-\sin\varphi$$

$$d_c^\infty = 1.58 + 4.09\tan^4\varphi$$

$$N_c = [e^{\pi\tan\varphi}\tan^2(\frac{1}{4}\pi+\frac{1}{2}\varphi)-1]\cot\varphi$$

$$K_q^0 = e^{(\frac{1}{2}\pi+\varphi)\tan\varphi}\cos\varphi\tan(\frac{1}{4}\pi+\frac{1}{2}\varphi)-e^{-(\frac{1}{2}\pi-\varphi)\tan\varphi}\cos\varphi\tan(\frac{1}{4}\pi-\frac{1}{2}\varphi)$$

$$K_q^\infty = N_c d_c^\infty K_o\tan\varphi$$

$$\alpha_q = \frac{K_q^0}{(K_q^\infty - K_q^0)}\frac{K_o\sin\varphi}{\sin(\frac{1}{4}\pi+\frac{1}{2}\varphi)}$$

$$K_q^z = \frac{K_q^0 + K_q^\infty\alpha_q\frac{z}{D}}{1+\alpha_q\frac{z}{D}}$$

$$K_c^0 = [e^{(\frac{1}{2}\pi+\varphi)\tan\varphi} \cos\varphi \tan(\frac{1}{4}\pi + \frac{1}{2}\varphi) - 1]\cot\varphi$$

$$K_c^\infty = N_c d_c^\infty$$

$$\alpha_c = \frac{K_c^0}{K_c^\infty - K_c^0} 2\sin(\frac{1}{4}\pi + \frac{1}{2}\varphi)$$

$$K_c^z = \frac{K_c^0 + K_c^\infty \alpha_c \frac{z}{D}}{1 + \alpha_c \frac{z}{D}}$$

where:

z – depth (metres)

D – pile diameter (metres)

φ – soil friction angle (degrees)

B4 TYPICAL SOIL PROPERTIES

The following tables give guidance on typical values of soil parameters for design purposes.

TABLE B3

TYPICAL PROPERTIES OF COHESIVE SOILS

Term	Shear strength, C_u (kPa)		Field guide to consistency (in unsaturated state)
	Unsaturated	Saturated	
Very soft	≤12	≤6	Exudes between fingers when squeezed in hand
Soft	12 to 25	6 to 12	Can be moulded by light finger pressure
Firm	25 to 50	12 to 25	Can be moulded by strong finger pressure
Stiff	50 to 100	25 to 50	Cannot be moulded by fingers. Can be indented by thumb
Very stiff	100 to 200	50 to 100	Can be indented by thumb nail
Hard	≥200	≥100	Can be indented with difficulty by thumb nail

TABLE B4

TYPICAL PROPERTIES OF NON-COHESIVE SOILS

Soil type	Angle of friction, ϕ (degrees)
Loose gravel with sand content	28° - 30°
Medium dense gravel with low sand content	30° - 36°
Dense to very dense gravel with low sand content	36° - 45°
Loose well graded sandy gravel	28° - 30°
Medium dense clayey sandy gravel	30° - 35°
Dense to very dense clayey sandy gravel	35° - 40°
Loose, coarse to fine sand	28° - 30°
Medium dense, coarse to fine sand	30° - 35°
Dense to very dense, coarse to fine sand	35° - 40°
Loose, fine and silty sand	28° - 30°
Medium dense, fine and silty sand	30° - 35°
Dense to very dense, fine and silty sand	35° - 40°

TABLE B5

TYPICAL SOIL DENSITIES

Soil type	Density (kN/m^3)	
	Unsaturated	Saturated
Cohesive soils	16 to 18	9 to 11
Non-cohesive soils:		
Gravel	16 to 20	9.5 to12.5
Coarse and medium sands	17 to 21	9.5 to 12.5
Fine and silty sands	17.5 to 21.5	9.5 to 12.5
Rock/soil mix—granite and shales	17.5 to 21	9.5 to 12.5
Rock/soil mix —basalts and dolerites	17.5 to 22.5	11 to 16
Rock/soil mix —limestones and sandstones	13 to 19	6.5 to 12.5

NOTE: The saturated densities given above result from influencing combinations of soil density reduction for submerged conditions and soil density increase due to soil porosity for the different soil types.

The density of non-cohesive materials should be determined in situ. For consistency of results, it is recommended that the test method from the appropriate Australian Standard be used to evaluate density (and other) soil parameters.

B5 REFERENCES

BRINCH HANSEN, J. and CHRISTENSEN, N. H. *The Ultimate Resistance of Rigid Piles Against Transverse Forces*. Bulletin No. 12 – Geoteknisk Institut: (Copenhagen 1961)

'92 AUSTROADS Bridge Design Code. Section 3: *Foundations.*

'92 AUSTROADS Bridge Design Code. Section 3: *Foundations—Commentary.*