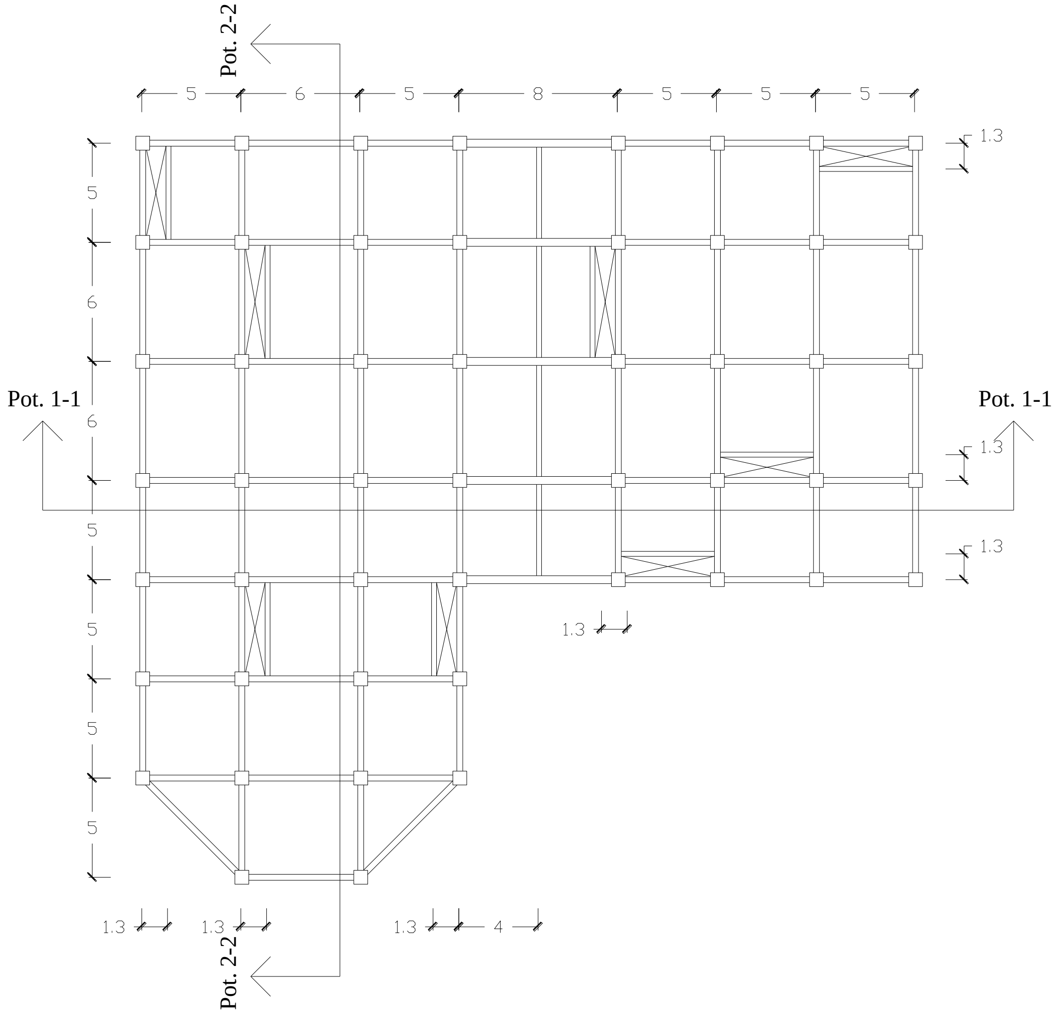
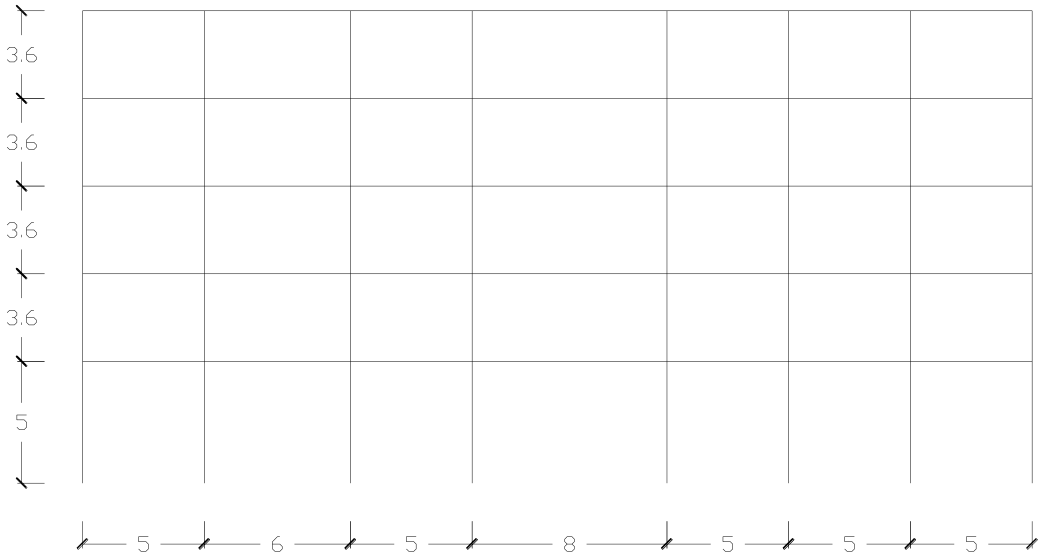
LAMPIRAN 1

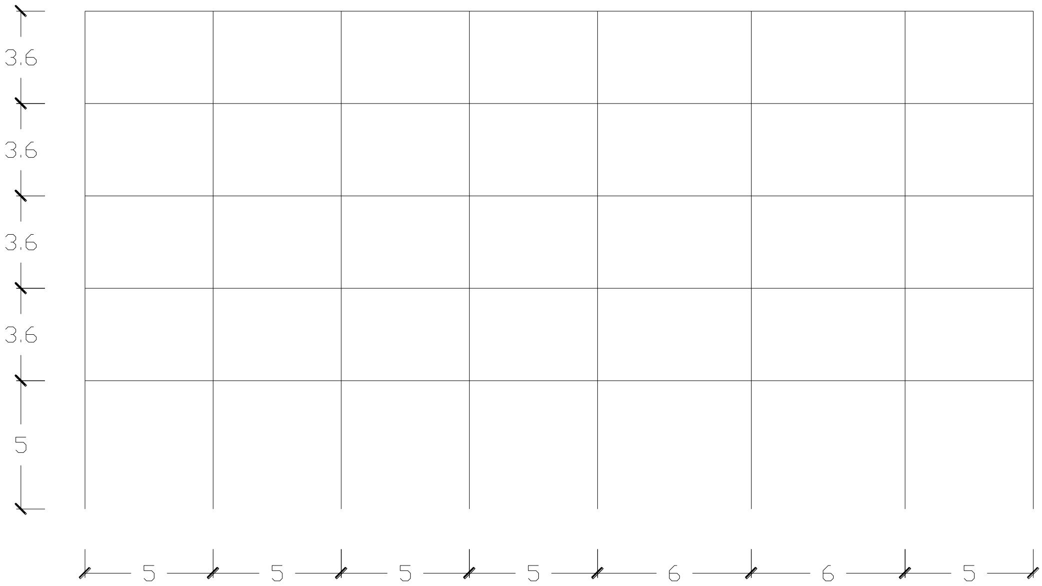
DENAH STRUKTUR



Gambar L1.1 Tampak Atas



Gambar L1.2 Potongan 1-1



Gambar L1.3 Potongan 2-2

LAMPIRAN 2

PRELIMINARY DESIGN

1) *Preliminary* Beban

Beban mati tambahan (*SDL*)

Berat sendiri pelat lantai (*DL*) : $0{,}2m \times 2400\, kg/m^3$	=	480	kg/m^2
Beban hidup (*LL*)	=	250	kg/m^2
Beban dinding	=	250	kg/m^2
Berat *finishing* (*SDL*)			
Adukan (3 cm) : $3 \times 21 kg/m^2$	=	63	kg/m^2
Keramik (2 cm) : $2 \times 25 kg/m^2$	=	50	kg/m^2
Berat palfon dan penggantung: $(7{,}5 + 15) kg/m^2$	=	22,5	kg/m^2
Mekanikal Elektrikal $20 kg/m^2$	=	20	kg/m^2 +
SDL	=	155,5	kg/m^2

2) *Preliminary* Balok

Dimensi balok ditentukan berdasarkan Tabel 8 SNI 03-2847-2002, hal 63.

Tabel L2.1 Tebal Minimum Balok Non-Prategang Atau Pelat Satu Arah Bila Lendutan Tidak Dihitung

	Tebal minimum, *h*			
Komponen struktur	Dua tumpuan sederhana	Satu ujung menerus	Kedua ujung menerus	Kantilever
	Komponen yang tidak menahan atau tidak disatukan dengan partisi atau konstruksi lain yang mungkin akan rusak oleh lendutan yang besar			
Pelat masif satu arah	$\frac{L}{20}$	$\frac{L}{24}$	$\frac{L}{28}$	$\frac{L}{10}$
Balok atau pelat rusuk satu arah	$\frac{L}{16}$	$\frac{L}{18{,}5}$	$\frac{L}{21}$	$\frac{L}{8}$

Tabel L2.1 Lanjutan
CATATAN Panjang bentang dalam mm Nilai yang diberikan harus digunakan langsung untuk komponen struktur dengan beton normal (Wc = 2400 kg/m3) dan tulangan BJTD 40. Untuk kodisi lain, nilai di atas harus dimodifikasikan sebagai berikut: (a) Untuk struktur beton rigan dengan berat jenis d antara 1500 kg/m3 sampai 2000 kg/m3, nilai tadi harus dikalikan dengan (1,65–0,0003Wc) tetapi tidak kurang dari 1,09, dimana Wc adalah berat jenis dalam kg/m3 (b) Untuk fy selain 400 Mpa, nilainya harus dikalikan dengan $0{,}4 + \frac{f_y}{700}$

<u>Balok 5 m</u>

$$h = \frac{L}{16} = \frac{5000}{16} = 312{,}5 \text{ mm} \sim 350 \text{ mm}$$

$$b = \frac{1}{2}h = \frac{1}{2}(350) = 175 \text{ mm} \sim 200 \text{ mm}$$

Digunakan balok ukuran 30 x 60 cm

<u>Balok 6 m</u>

$$h = \frac{L}{16} = \frac{6000}{16} = 375 \text{ mm} \sim 400 \text{ mm}$$

$$b = \frac{1}{2}h = \frac{1}{2}(375) = 187{,}5 \text{ mm} \sim 200 \text{ mm}$$

Digunakan balok ukuran 30 x 60 cm

<u>Balok 8 m</u>

$$h = \frac{L}{16} = \frac{8000}{16} = 500 \text{ mm} \sim 500 \text{ mm}$$

$$b = \frac{1}{2}h = \frac{1}{2}(500) = 250 \text{ mm} \sim 250 \text{ mm}$$

Digunakan balok ukuran 40 x 70 cm

3) *Preliminary* Pelat

Berdasarkan SNI 03-2847-2002, hal 65 pasal (3) Tebal pelat minimum dengan balok yang menghubungkan tumpuan pada semua sisinya harus memenuhi ketentuan sebagai berikut:

(a) Untuk α_m yang sama atau lebih kecil dari 0,2, harus menggunakan 11.5(3(2)).

(b) Untuk α_m lebih besar dari 0,2 tapi tidak lebih dari 2,0, ketebalan pelat minimum harus memenuhi:

$$h = \frac{L_n\left(0{,}8 + \frac{f_y}{1500}\right)}{36 + 5\beta(\alpha_m - 0{,}2)}$$

dan tidak boleh kurang dari 120 mm.

(c) Untuk α_m lebih besar dari 0,2, ketebalan pelat minimum tidak boleh kurang dari:

$$h = \frac{L_n\left(0{,}8 + \frac{f_y}{1500}\right)}{36 + 9\beta}$$

(d) Pada tepi yang tidak menerus, balok tepi harus mempunyai rasio kekakuan α tidak kurang dari 0,8 atau sebagai altrnatif ketebalan minimum yang ditentukan persamaan (16) atau persamaan (17) harus dinaikkan paling tidak 10% pada panel dengan tepi yang tidak menerus.

Perhitungan tebal pelat mengambil panel yang terbesar dari area atap yaitu pelat yang berukuran 8 x 6 m dengan asumsi $\alpha_m \geq 2$:

Bentang bersih : $L_{npendek}$ = 5000 mm

$L_{npanjang}$ = 6000 mm

Ukuran balok : 400 x 700 mm

$$\beta = \frac{L_{n\ panjang}}{L_{n\ pendek}} = \frac{6000}{5000}$$

$$= 1{,}2 < 2 \rightarrow \text{pelat 2 arah}$$

Penentuan tebal pelat 2 arah

$$h_{min} = \frac{L_n\left(0{,}8 + \frac{f_y}{1500}\right)}{36 + 9\beta} = \frac{5000\left(0{,}8 + \frac{400}{1500}\right)}{36 + 9(1{,}2)} = 113{,}9601 \text{ mm}$$

$$h_{max} = \frac{L_n\left(0{,}8+\frac{f_y}{1500}\right)}{36+9\beta} = \frac{6000\left(0{,}8+\frac{400}{1500}\right)}{36+9(1{,}2)}$$

$= 136{,}7521$ mm

Maka tebal pelat atap (h_{atap}) = 200 mm

Perhitungan tebal pelat mengambil panel yang terbesar dari area lantai yaitu pelat yang berukuran 8 x 6 m dengan asumsi $\alpha_m \geq 2$:

Ukuran balok : 250 x 500 mm

Bentang bersih : $L_{npendek}$ = 5000 mm

$L_{npanjang}$ = 6000 mm

$$\beta = \frac{L_{n\ panjang}}{L_{n\ pendek}} = \frac{6000}{5000}$$

$= 1{,}2 < 2 \rightarrow$ pelat 2 arah

Penentuan tebal pelat 2 arah

$$h_{min} = \frac{L_n\left(0{,}8+\frac{f_y}{1500}\right)}{36+9\beta} = \frac{5000\left(0{,}8+\frac{400}{1500}\right)}{36+9(1{,}2)}$$

$= 113{,}9601$ mm

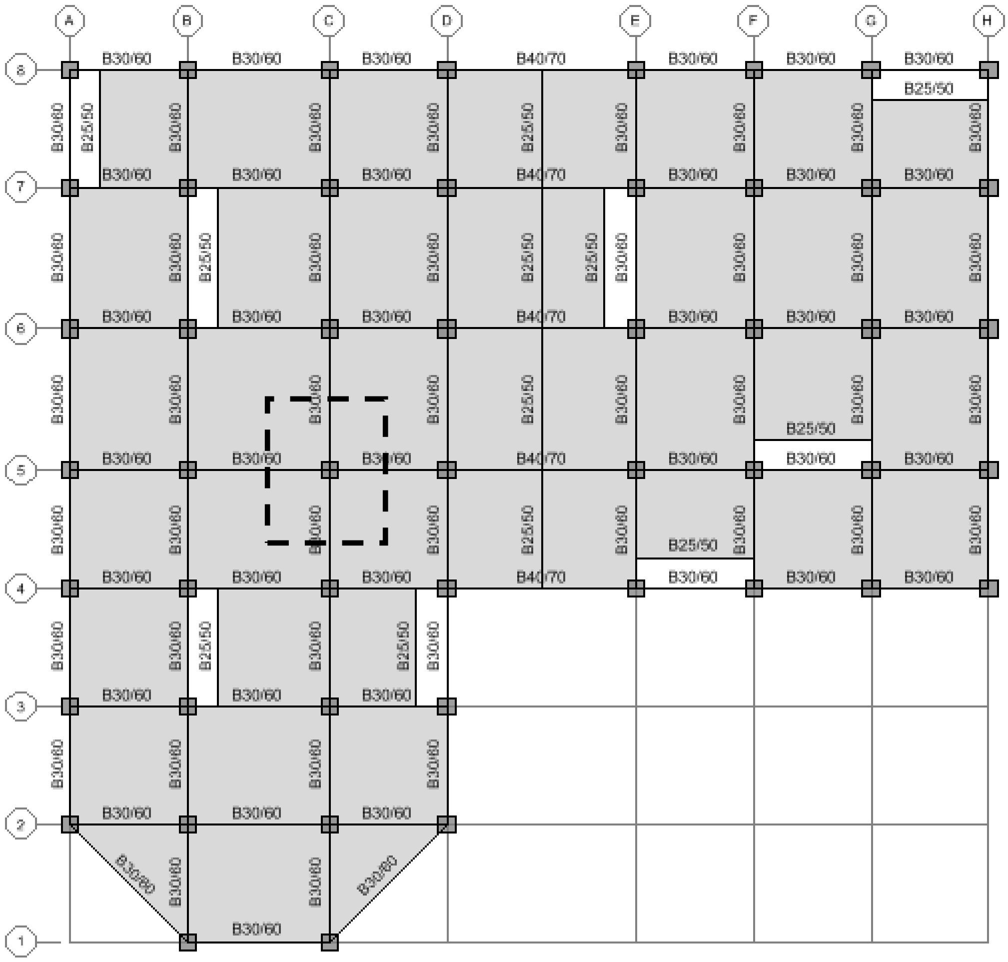
$$h_{max} = \frac{L_n\left(0{,}8+\frac{f_y}{1500}\right)}{36+9\beta} = \frac{6000\left(0{,}8+\frac{400}{1500}\right)}{36+9(1{,}2)}$$

$= 136{,}7521$ mm

Maka tebal pelat atap (h_{lantai}) = 150 mm

4) *Preliminary* Kolom

Dimensi kolom diperoleh berdasarkan kondisi pembebanan yang bekerja pada kolom yaitu distribusi pembebanan dari pelat dan balok. Kolom harus direncanakan untuk memikul beban aksial yang bekerja pada semua lantai atau atap dan momen maksmum yang berasal dari beban terfaktor pada satu bentang terdekat dari lantai atau atap yang ditinjau. Kombinasi pembebanan yang menghasilkan rasio maksimum dari momen terhadap beban aksial juga harus diperhitungkan. Momen-momen yang bekerja pada setiap level lantai atau atap harus didistribusikan pada kolom di atas dan di bawah lantai tersebut berdasarkan kekakuan relatif kolom.



Gambar L2.1 Perencanaan Kolom

Beban mati pada pelat, t_{atap} = 150 mm, t_{lantai} = 200 mm

$$
\begin{aligned}
A_{pelat\ 1} &= 3m \times 3m &&= 9 \quad m^2 \\
A_{pelat\ 2} &= 3m \times 2{,}5m \times 2 &&= 15 \quad m^2 \\
A_{pelat\ 3} &= 2{,}5m \times 2{,}5m &&= 6{,}25 \quad m^2 \quad + \\
& & A &= 30{,}25 \quad m^2
\end{aligned}
$$

Kolom Lantai 5

Berdasarkan Gambar L2.1, perhitungan beban kolom lantai 5 adalah sebagai berikut:

Berat sendiri pelat (*DL*) : $0{,}15m \times 30{,}25m^2 \times 2400\,kg/m^3$	=	10890	kg	
Adukan (3 cm) : $3 \times 30{,}25m^2 \times 21kg/m^2$	=	1905,75	kg	
Keramik : $30{,}25m^2 \times 25kg/m^2$	=	756,25	kg	
Plafon *gypsum* : $30{,}25m^2 \times 7{,}5kg/m^2$	=	226,875	kg	
Rangka plafon : $30{,}25m^2 \times 15kg/m^2$	=	453,75	kg	
Mekanikal Elektrikal : $30{,}25m^2 \times 20kg/m^2$	=	605	kg	+
SDL	=	14837,625	kg	

Beban mati pada balok

Berat sendiri balok (B1 : 300 x 600)				
$0{,}3 \times 0{,}6 \times 11m \times 2400\,kg/m^3$	=	4752	kg	
Dinding, tinggi 3,6 m				
$3{,}6m \times 11m \times 250\,kg/m^2$	=	9900	kg	+
DL	=	14652	kg	

Beban hidup bangunan = 250 kg/m^2

q_{LL} = $30{,}25m^2 \times 250\,kg/m^2$

= 7562,5 kg

Beban *ultimate* (q_u)

q_u = $1{,}2(DL + SDL) + 1{,}6LL$

= $1{,}2(14652 + 14837{,}625) + 1{,}6(7562{,}5)$

= 47487,55 kg

Desain dimensi kolom untuk perencanaannya dapat digunakan asumsi bahwa kolom harus kuat menahan beban mati dengn 40% kekuatan tekannya, sehingga dapat digunakan persamaan sebagai berikut:

$$A = \frac{\sum DL}{40\% \times 0,85 f_c'}$$

$$= \frac{370521,25}{0,4 \times 0,85 \times 25}$$

$$= 43590,73529 \text{ mm}^2$$

Dimana, ΣDL = Total beban mati dan hidup yang paling besar ditanggung oleh kolom paling bawah

= 14652 + 14837,625 + 7562,5

= 37052,125 kg

= 370521,25 N

A = Luas penampang kolom

Asumsi $b = h \rightarrow A = b \times h$

$$43590,73529 = h \times h$$

$$h^2 = 43590,73529$$

$$b = h = 208,7839 \text{ mm}$$

Jadi diambil dimensi kolom untuk lantai 5 adalah 600×600 mm

<u>Kolom Lantai 4</u>

Berdasarkan gambar L2.1, perhitungan beban kolom lantai 4 adalah sebagai berikut:

Berat *SDL* lantai 5	=	14837,625	kg
Berat sendiri pelat (*DL*) : $0,2m \times 30,25m^2 \times 2400\,kg/m^3$	=	14520	kg
Adukan (3 cm) : $3 \times 30,25m^2 \times 21kg/m^2$	=	1905,75	kg
Keramik : $30,25m^2 \times 25kg/m^2$	=	756,25	kg
Plafon *gypsum* : $30,25m^2 \times 7,5kg/m^2$	=	226,875	kg
Rangka plafon : $30,25m^2 \times 15kg/m^2$	=	453,75	kg
Mekanikal Elektrikal : $30,25m^2 \times 20kg/m^2$	=	605	kg +
SDL	=	33305,25	kg

Beban mati pada balok

Berat sendiri balok (B1 : 300 x 600)

$0{,}3 \times 0{,}6 \times 11m \times 2400\,kg/m^3$	=	4752	kg	
Dinding, tinggi 3,6 m				
$3{,}6m \times 11m \times 250\,kg/m^2$	=	9900	kg	
Berat kolom lantai 5				
$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg	+
DL	=	17762,4	kg	

Beban hidup bangunan = 250 kg/m^2

q_{LL} = $30{,}25m^2 \times 250\,kg/m^2$

= 7562,5 kg

Beban *ultimate* (q_u)

q_u = $1{,}2(DL + SDL) + 1{,}6LL$

= $1{,}2(17762{,}4 + 33305{,}25) + 1{,}6(7562{,}5)$

= 73381,18 kg

Desain dimensi kolom untuk perencanaannya dapat digunakan asumsi bahwa kolom harus kuat menahan beban mati dengn 40% kekuatan tekannya, sehingga dapat digunakan persamaan sebagai berikut:

$$A = \frac{\sum DL}{40\% \times 0{,}85 f_c'} = \frac{586301{,}5}{0{,}4 \times 0{,}85 \times 25}$$

= 68976,64706 mm^2

Dimana, ΣDL = Total beban mati dan hidup yang paling besar ditanggung oleh kolom paling bawah

= 17762,4 + 33305,25 + 7562,5

= 58630,15 kg

= 586301,5 N

$A \quad = \quad$ Luas penampang kolom

Asumsi $\quad b = h \rightarrow \quad A = b \times h$

$$68976{,}64706 = h \times h$$

$$h^2 = 68976{,}64706$$

$$b = h = 262{,}6341 \text{ mm}$$

Jadi diambil dimensi kolom untuk lantai 4 adalah 600×600 mm

Kolom Lantai 3

Berdasarkan gambar L2.1, perhitungan beban kolom lantai 3 adalah sebagai berikut:

Berat *SDL* lantai 4	=	33305,25	kg
Berat sendiri pelat (*DL*) : $0{,}2m \times 30{,}25m^2 \times 2400\,kg/m^3$	=	14520	kg
Adukan (3 cm) : $3 \times 30{,}25m^2 \times 21kg/m^2$	=	1905,75	kg
Keramik : $30{,}25m^2 \times 25kg/m^2$	=	756,25	kg
Plafon *gypsum* : $30{,}25m^2 \times 7{,}5kg/m^2$	=	226,875	kg
Rangka plafon : $30{,}25m^2 \times 15kg/m^2$	=	453,75	kg
Mekanikal Elektrikal : $30{,}25m^2 \times 20kg/m^2$	=	605	kg +
SDL	=	51772,875	kg

Beban mati pada balok

Berat sendiri balok (B1 : 300 x 600)

$0{,}3 \times 0{,}6 \times 11m \times 2400\,kg/m^3$	=	4752	kg

Dinding, tinggi 3,6 m

$3{,}6m \times 11m \times 250\,kg/m^2$	=	9900	kg

Berat kolom lantai 5

$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg

Berat kolom lantai 4

$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg +
DL	=	20872,8	kg

Beban hidup bangunan = 250 kg/m^2

$$q_{LL} = 30{,}25 m^2 \times 250\, kg/m^2$$

$$= 7562{,}5 \text{ kg}$$

Beban *ultimate* (q_u)

$$q_u = 1{,}2(DL + SDL) + 1{,}6LL$$

$$= 1{,}2(20872{,}8 + 51772{,}875) + 1{,}6(7562{,}5)$$

$$= 99274{,}81 \text{ kg}$$

Desain dimensi kolom untuk perencanaannya dapat digunakan asumsi bahwa kolom harus kuat menahan beban mati dengn 40% kekuatan tekannya, sehingga dapat digunakan persamaan sebagai berikut:

$$A = \frac{\sum DL}{40\% \times 0{,}85 f_c'}$$

$$= \frac{802081{,}75}{0{,}4 \times 0{,}85 \times 25}$$

$$= 94362{,}55882 \text{ mm}^2$$

Dimana, ΣDL = Total beban mati dan hidup yang paling besar ditanggung oleh kolom paling bawah

= 20872,8 + 33305,25 + 7562,5

= 80208,175 kg

= 802081,75 N

A = Luas penampang kolom

Asumsi $b = h \rightarrow A = b \times h$

$$94362{,}55882 = h \times h$$

$$h^2 = 94362{,}55882$$

$$b = h = 307{,}1849 \text{ mm}$$

Jadi diambil dimensi kolom untuk lantai 3 adalah 700×700 mm

Kolom Lantai 2

Berdasarkan gambar L2.1, perhitungan beban kolom lantai 2 adalah sebagai berikut:

Berat *SDL* lantai 3	=	51772,875	kg	
Berat sendiri pelat (*DL*) : $0{,}2m \times 30{,}25m^2 \times 2400\,kg/m^3$	=	14520	kg	
Adukan (3 cm) : $3 \times 30{,}25m^2 \times 21kg/m^2$	=	1905,75	kg	
Keramik : $30{,}25m^2 \times 25kg/m^2$	=	756,25	kg	
Plafon *gypsum* : $30{,}25m^2 \times 7{,}5kg/m^2$	=	226,875	kg	
Rangka plafon : $30{,}25m^2 \times 15kg/m^2$	=	453,75	kg	
Mekanikal Elektrikal : $30{,}25m^2 \times 20kg/m^2$	=	605	kg	+
SDL	=	70240,5	kg	

Beban mati pada balok

Berat sendiri balok (B1 : 300 x 600)				
$0{,}3 \times 0{,}6 \times 11m \times 2400\,kg/m^3$	=	4752	kg	
Dinding, tinggi 3,6 m				
$3{,}6m \times 11m \times 250\,kg/m^2$	=	9900	kg	
Berat kolom lantai 5				
$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg	
Berat kolom lantai 4				
$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg	
Berat kolom lantai 3				
$3{,}6m \times 0{,}7m \times 0{,}7m \times 2400\,kg/m^3$	=	4233,6	kg	+
DL	=	25106,4	kg	

Beban hidup bangunan = 250 kg/m^2

q_{LL} = $30{,}25m^2 \times 250\,kg/m^2$

= 7562,5 kg

Beban *ultimate* (q_u)

$$q_u = 1{,}2(DL + SDL) + 1{,}6LL$$

$$= 1{,}2(25106{,}4 + 70240{,}5) + 1{,}6(7562{,}5)$$

$$= 126516{,}28 \text{ kg}$$

Desain dimensi kolom untuk perencanaannya dapat digunakan asumsi bahwa kolom harus kuat menahan beban mati dengn 40% kekuatan tekannya, sehingga dapat digunakan persamaan sebagai berikut:

$$A = \frac{\sum DL}{40\% \times 0{,}85 f_c'}$$

$$= \frac{1029094}{0{,}4 \times 0{,}85 \times 25}$$

$$= 121069{,}8824 \text{ mm}^2$$

Dimana, ΣDL = Total beban mati dan hidup yang paling besar ditanggung oleh kolom paling bawah

= 25106,4 + 70240,5 + 7562,5

= 102909,4 kg

= 1029094 N

A = Luas penampang kolom

Asumsi $b = h \rightarrow A = b \times h$

$$121069{,}8824 = h \times h$$

$$h^2 = 121069{,}8824$$

$$b = h = 347{,}951 \text{ mm}$$

Jadi diambil dimensi kolom untuk lantai 2 adalah 700×700 mm

<u>Kolom Lantai 1</u>

Berdasarkan gambar L2.1, perhitungan beban kolom lantai 1 adalah sebagai berikut:

Berat *SDL* lantai 2	=	70240,5	kg	
Berat sendiri pelat (*DL*) : $0{,}2m \times 30{,}25m^2 \times 2400\,kg/m^3$	=	14520	kg	
Adukan (3 cm) : $3 \times 30{,}25m^2 \times 21kg/m^2$	=	1905,75	kg	
Keramik : $30{,}25m^2 \times 25kg/m^2$	=	756,25	kg	
Plafon *gypsum* : $30{,}25m^2 \times 7{,}5kg/m^2$	=	226,875	kg	
Rangka plafon : $30{,}25m^2 \times 15kg/m^2$	=	453,75	kg	
Mekanikal Elektrikal : $30{,}25m^2 \times 20kg/m^2$	=	605	kg	+
SDL	=	88708,125	kg	

Beban mati pada balok

Berat sendiri balok (B1 : 300 x 600)				
$0{,}3 \times 0{,}6 \times 11m \times 2400\,kg/m^3$	=	4752	kg	
Dinding, tinggi 3,6 m				
$5m \times 11m \times 250\,kg/m^2$	=	13750	kg	
Berat kolom lantai 5				
$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg	
Berat kolom lantai 4				
$3{,}6m \times 0{,}6m \times 0{,}6m \times 2400\,kg/m^3$	=	3110,4	kg	
Berat kolom lantai 3				
$3{,}6m \times 0{,}7m \times 0{,}7m \times 2400\,kg/m^3$	=	4233,6	kg	
Berat kolom lantai 2				
$3{,}6m \times 0{,}7m \times 0{,}7m \times 2400\,kg/m^3$	=	4233,6	kg	+
DL	=	33190	kg	

Beban hidup bangunan = 250 kg/m^2

q_{LL} = $30{,}25m^2 \times 250\,kg/m^2$

= 7562,5 kg

Beban *ultimate* (q_u)

$$q_u = 1{,}2(DL + SDL) + 1{,}6LL$$

$$= 1{,}2(33190 + 88708{,}125) + 1{,}6(7562{,}5)$$

$$= 158377{,}75 \text{ kg}$$

Desain dimensi kolom untuk perencanaannya dapat digunakan asumsi bahwa kolom harus kuat menahan beban mati dengn 40% kekuatan tekannya, sehingga dapat digunakan persamaan sebagai berikut:

$$A = \frac{\sum DL}{40\% \times 0{,}85 f_c'}$$

$$= \frac{1294606{,}25}{0{,}4 \times 0{,}85 \times 25}$$

$$= 152306{,}6176 \text{ mm}^2$$

Dimana, ΣDL = Total beban mati dan hidup yang paling besar ditanggung oleh kolom paling bawah

= 33190 + 88708,125 + 7562,5

= 129460,625 kg

= 1294606,25 N

A = Luas penampang kolom

Asumsi $b = h \rightarrow A = b \times h$

$$152306{,}6176 = h \times h$$

$$h^2 = 152306{,}6176$$

$$b = h = 390{,}2648 \text{ mm}$$

Jadi diambil dimensi kolom untuk lantai 1 adalah 700×700 mm

LAMPIRAN 3

LENDUTAN PADA BALOK

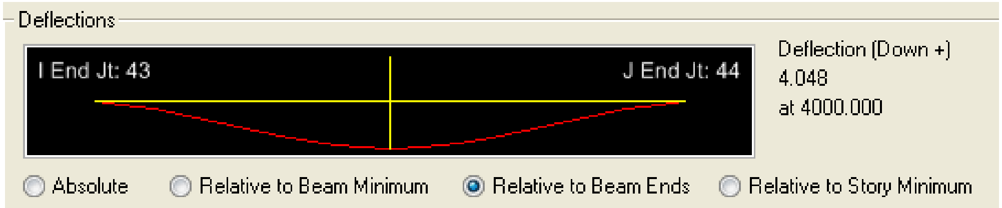
Perbandingan antara lendutan aktual yang diperoleh dari hasil *output* dengan lendutan ijin maksimum berdasarkan SNI 03-2847-2002, hal 65 Tabel 9 dapat dilihat di bawah ini.

Tabel L3.1 Lendutan Izin Maksimum

<table>
<tr><th>Jenis komponen struktur</th><th>Lendutan yang diperhitungkan</th><th>Batas lendutan</th></tr>
<tr><td>Atap datar yang tidak menahan atau tidak disatukan dengan komponen nonstructural yang mungkin akan rusak oleh lendutan yang besar</td><td>Lendutan seketika akibat beban hidup (L)</td><td>$\frac{L^{a}}{180}$</td></tr>
<tr><td>Lantai yang tidak menahan atau tidak disatukan dengan komponen nonstructural yang mungkin akan rusak oleh lendutan yang besar</td><td>Lendutan seketika akibat beban hidup (L)</td><td>$\frac{L}{360}$</td></tr>
<tr><td>Konstruksi atap atau lantai yang menahan atau disatukan dengan komponen nonstruktural yang mungkin akan rusak oleh lendutan yang besar</td><td rowspan="2">Bagian dari lendutan total yang terjadi setelah pemasangan komponen nonstruktural (jumlah dari lendutan jangka panjang, akibat semua beban tetap yang bekerja, dan lendutan seketika, akibat penambahan beban hidup)[c]</td><td>$\frac{L^{b}}{180}$</td></tr>
<tr><td>Konstruksi atap atau lantai yang menahan atau disatukan dengan komponen nonstruktural yang mungkin tidak akan rusak oleh lendutan yang besar.</td><td>$\frac{L^{c}}{180}$</td></tr>
<tr><td colspan="3">[a] Batasan ini tidak dimaksudkan untuk mencegah kemungkinan penggenangan air. Kemungkinan penggenangan air harus diperiksa dengan melakukan perhitungan lendutan, termasuk lendutan tambahan akibat adanya penggenangan air tersebut, dan mempertimbangkan pengaruh jangka panjang dari beban yang selalu bekerja, lawan lendut, toleransi konstruksi dan keandalan sistem drainase.
[b] Batas lendutan boleh dilampaui bila langkah pencegahan kerusakan terhadap komponen yang ditumpu atau yang disatukan telah dilakukan.
[c] Lendutan jangka panjang harus dihitung berdasarkan ketentuan 11.5(2(5)) atau 11.5(4(2)), tetapi boleh dikurangi dengan nilai lendutan yang terjadi sebelum penambahan komponen non-struktural. Besarnya nilai lendutan ini harus ditentukan berdasarkan data teknis yang dapat diterima berkenaan dengan karakteristik hubungan waktu dan lendutan dari komponen struktur yang serupa dengan komponen struktur yang ditinjau.</td></tr>
</table>

Tabel L3.1 Lanjutan
[d] Tetapi tidak boleh lebih besar dari toleransi yang disediakan untuk komponen non-struktur. Batasan ini boleh dilampaui bila ada lawan lendut yang disediakan sedemikian hingga lendutan total dikurangi lawan lendut tidak melebihi batas lendutan yang ada.

Perbandingan antara lendutan aktual yang diperoleh dari hasil *output* dengan lendutan ijin maksimum berdasarkan SNI 03-2847-2002 adalah $\frac{L}{360}$ = 22,22 mm, dimana balok yang ditinjau memiliki panjang bentang 8 m (8000 mm). Hasil perhitungan analisis menunjukkan bahwa lendutan yang terjadi (tinjauan balok B45) adalah 4,048 mm. Maka lendutan memenuhi syarat.



Dalam satuan mm

Gambar L3.1 Lendutan Balok

LAMPIRAN 4

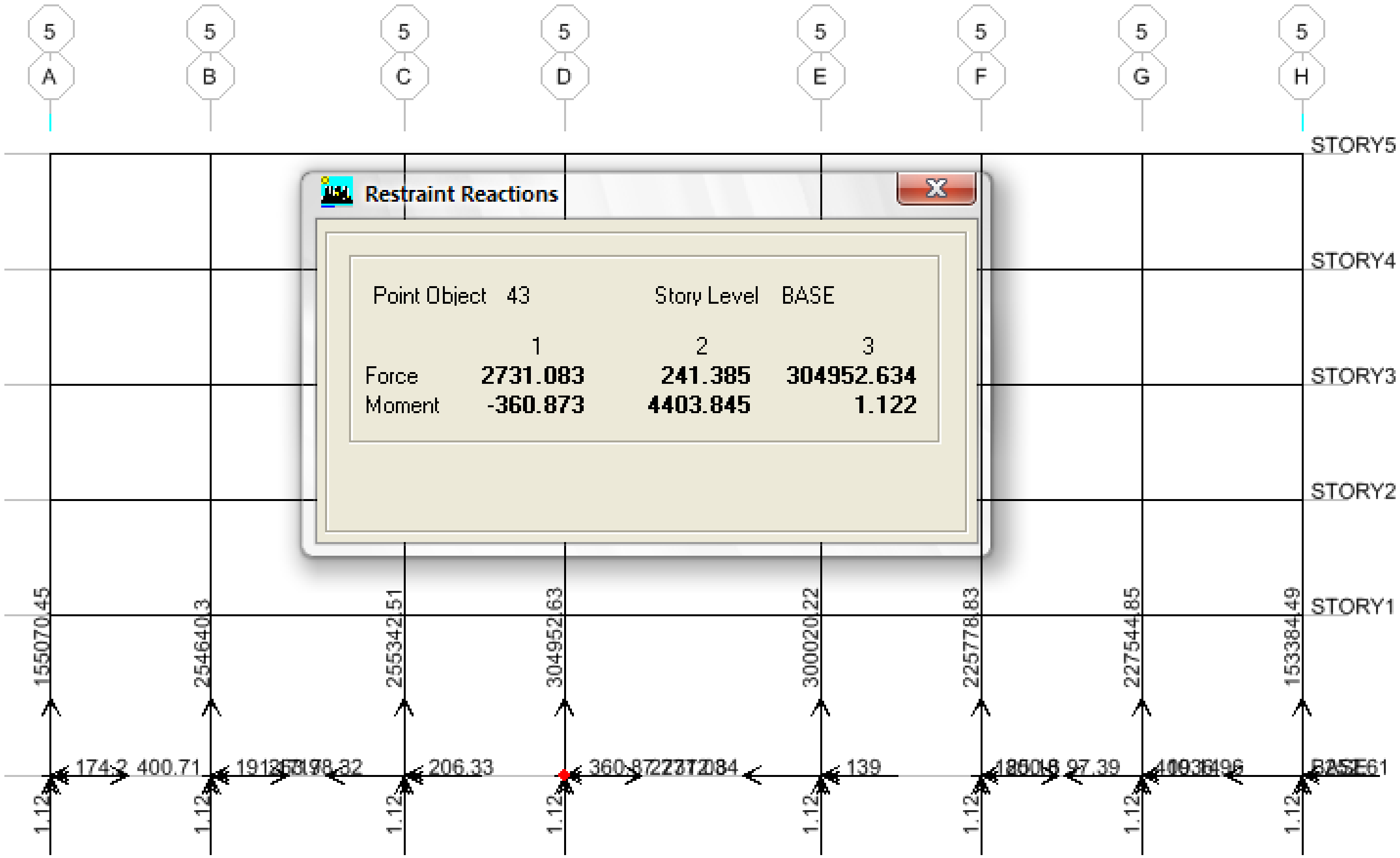
NILAI PERIODE GETAR

Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
1	0.70124	0.0465	82.6785	0	0.0465	82.6785	0	95.4376	0.0537	3.5135	95.4376	0.0537	3.5135
2	0.69021	86.3826	0.0703	0	86.4292	82.7488	0	0.081	99.4906	0.0389	95.5186	99.5444	3.5525
3	0.64635	0.071	3.6114	0	86.5001	86.3602	0	4.1155	0.08	82.5429	99.634	99.6244	86.0954
4	0.2237	0.0011	9.6903	0	86.5013	96.0505	0	0.1967	0	0.255	99.8308	99.6244	86.3504
5	0.2213	9.8276	0.0015	0	96.3289	96.052	0	0	0.2209	0.0002	99.8308	99.8453	86.3506
6	0.20739	0.0019	0.211	0	96.3308	96.263	0	0.0092	0.0001	9.7948	99.84	99.8454	96.1454
7	0.11629	0.0028	2.6781	0	96.3335	98.9412	0	0.1482	0.0001	0.0424	99.9882	99.8455	96.1877
8	0.11539	2.6506	0.0036	0	98.9842	98.9448	0	0.0002	0.1431	0.0027	99.9884	99.9887	96.1904
9	0.10806	0.006	0.0244	0	98.9902	98.9692	0	0.001	0.0004	2.7491	99.9894	99.9891	98.9395
10	0.07539	0.0009	0.7655	0	98.991	99.7347	0	0.0022	0	0.0016	99.9917	99.9891	98.9411
11	0.07517	0.7442	0.0009	0	99.7353	99.7357	0	0	0.0028	0.0035	99.9917	99.9919	98.9446
12	0.06984	0.0049	0	0	99.7401	99.7357	0	0.0001	0	0.7869	99.9918	99.9919	99.7315
13	0.05273	0.0093	0.2542	0	99.7494	99.9899	0	0.0079	0.0003	0.0001	99.9997	99.9922	99.7316
14	0.05269	0.2486	0.0098	0	99.998	99.9997	0	0.0003	0.0078	0.0016	100	99.9999	99.7331
15	0.0488	0.002	0.0003	0	100	100	0	0	0.0001	0.2669	100	100	100

LAMPIRAN 5

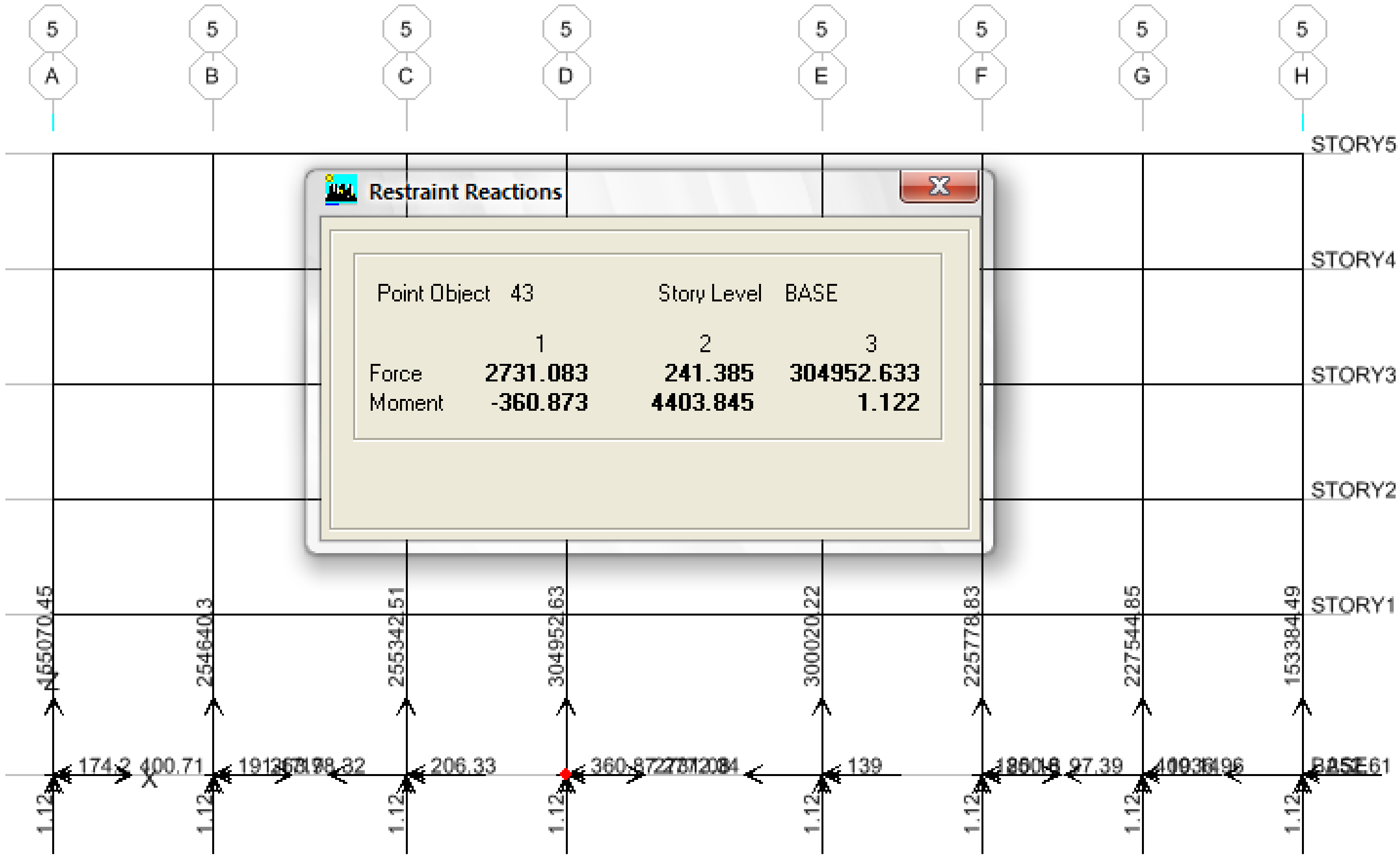
REAKSI PERLETAKAN

1. Gedung A



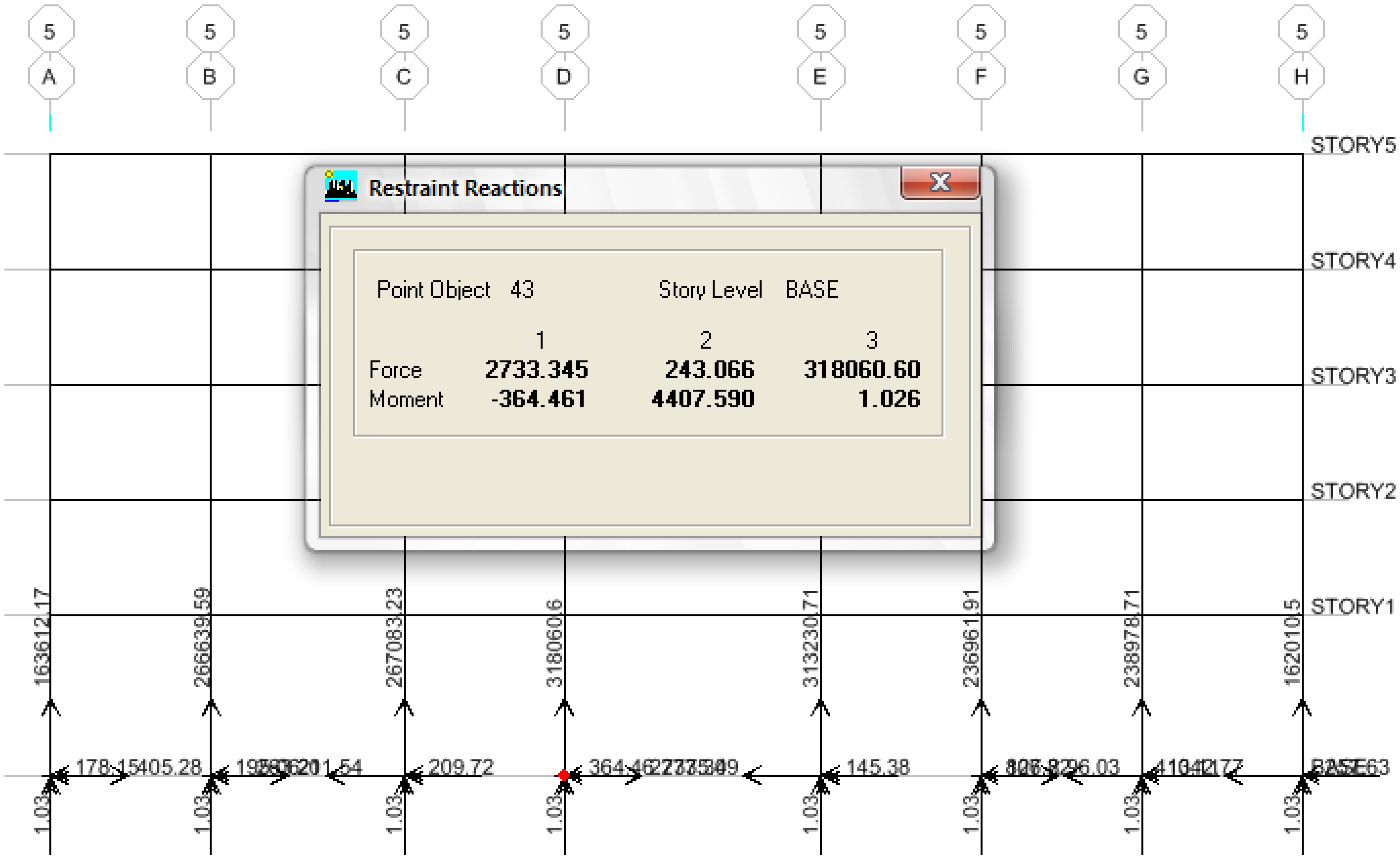
Gambar L5.1 Hasil Reaksi Perletakan untuk Metode A

2. Gedung B



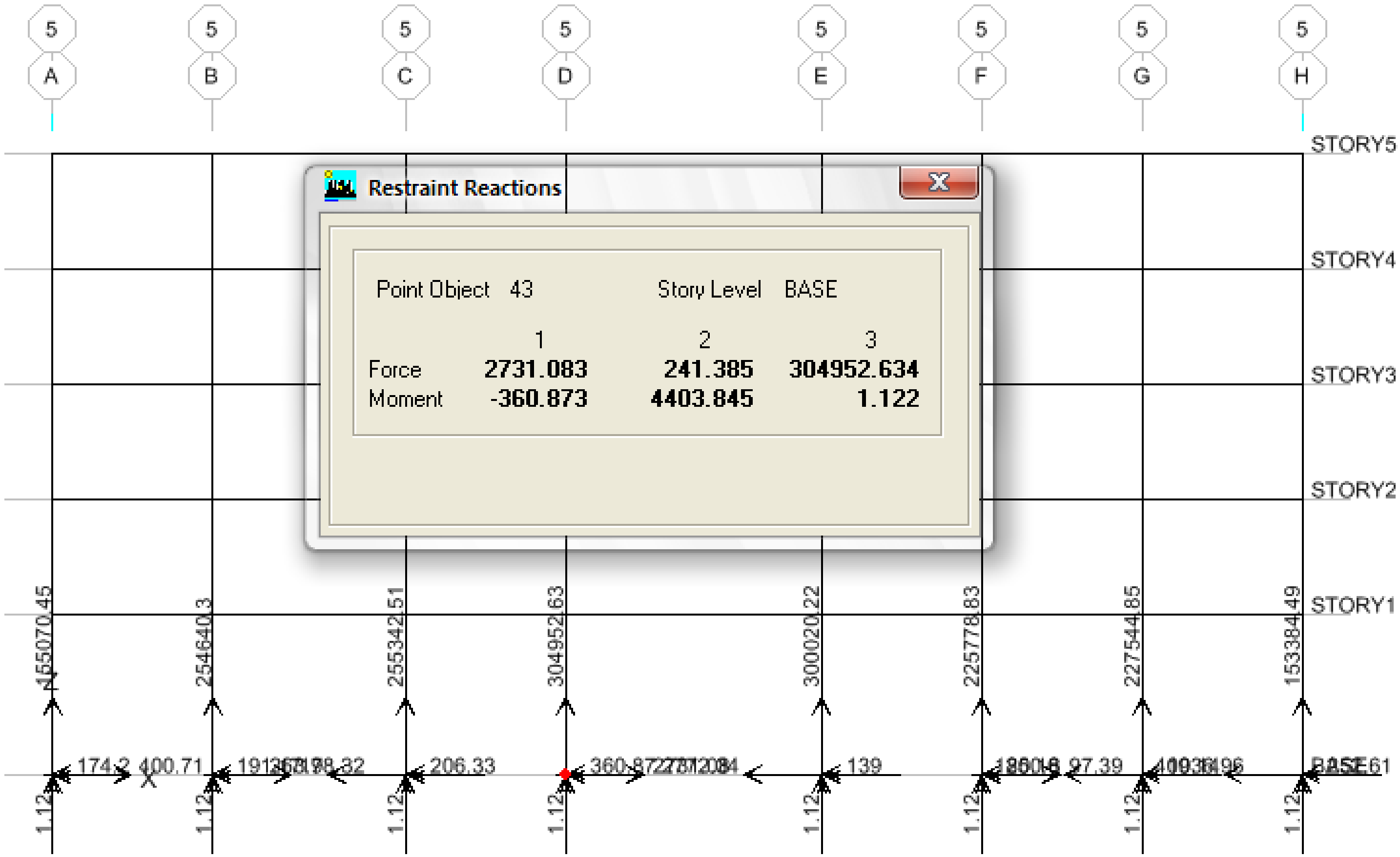
Gambar L5.2 Hasil Reaksi Perletakan untuk Metode B

3. Gedung C



Gambar L5.3 Hasil Reaksi Perletakan untuk Metode C

4. Gedung D



Gambar L5.4 Hasil Reaksi Perletakan untuk Metode D

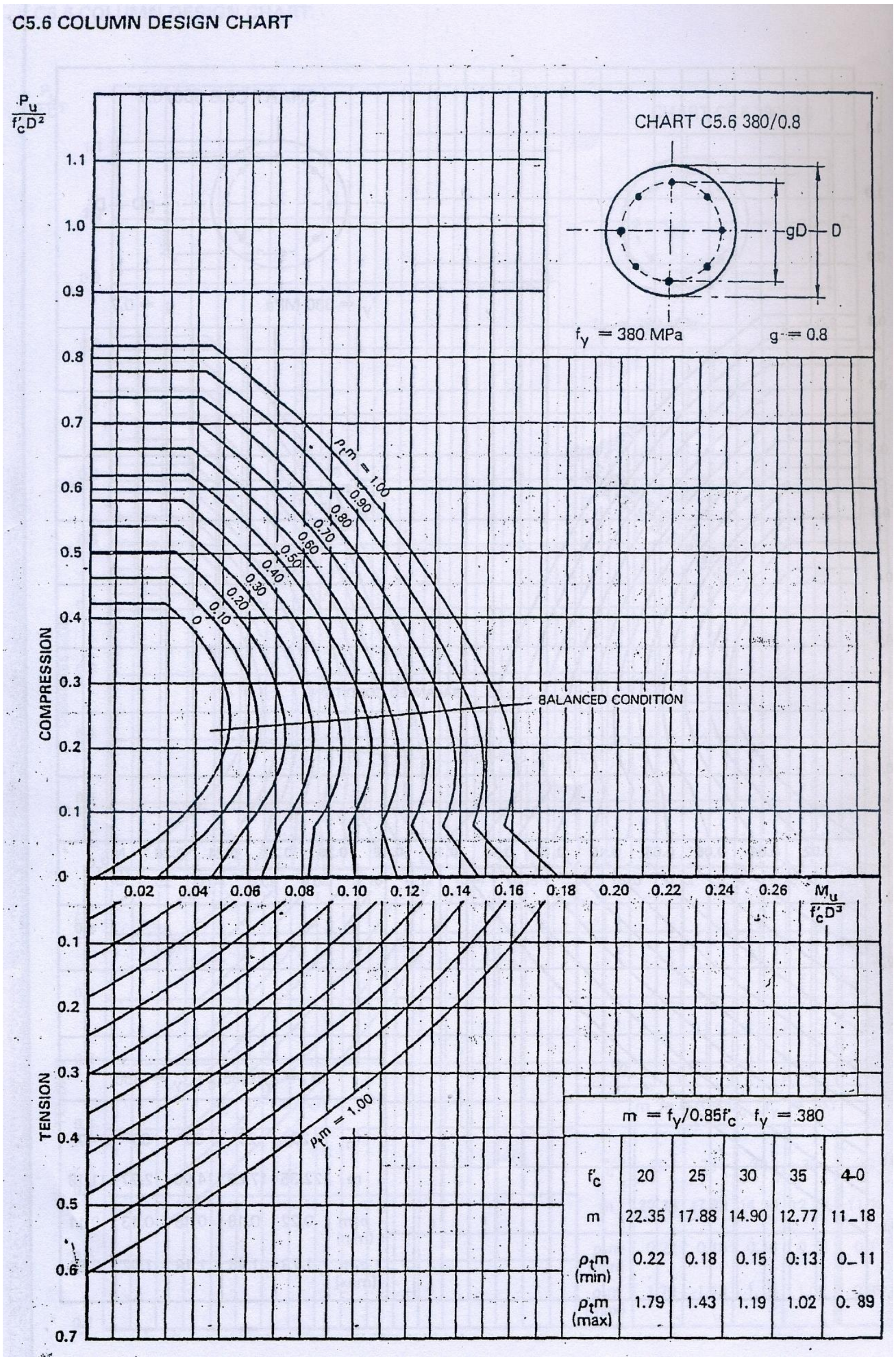
LAMPIRAN 6
LUAS TULANGAN BERULIR

Tabel 6.1 Luas Tulangan Berulir

Jumlah batang	**Diameter (mm)**								
	10	**12**	**13**	**14**	**16**	**18**	**19**	**22**	**25**
1	78.6	113.1	132.8	154.0	201.1	254.6	283.6	380.3	491.1
2	157.1	226.3	265.6	154.0	402.3	509.1	567.3	760.6	982.1
3	235.7	339.4	398.4	154.0	603.4	763.7	850.9	1140.9	1473.2
4	314.3	452.6	531.1	154.0	804.6	1018.3	1134.6	1521.1	1964.3
5	392.9	565.7	663.9	154.0	1005.7	1272.9	1418.2	1901.4	2455.4
6	471.4	678.9	796.7	154.0	1206.9	1527.4	1701.9	2281.7	2946.4
7	550.0	792.0	929.5	154.0	1408.0	1782.0	1985.5	2662.0	3437.5
8	628.6	905.1	1062.3	154.0	1609.1	2036.6	2269.1	3042.3	3928.6
9	707.1	1018.3	1195.1	154.0	1810.3	2291.1	2552.8	3422.6	4419.6
10	785.7	1131.4	1327.9	154.0	2011.4	2545.7	2836.4	3802.9	4910.7
11	864.3	1244.6	1460.6	154.0	2212.6	2800.3	3120.1	4183.1	5401.8
12	942.9	1357.7	1593.4	154.0	2413.7	3054.9	3403.7	4563.4	5892.9
13	1021.4	1470.9	1726.2	154.0	2614.9	3309.4	3687.4	4943.7	6383.9
14	1100.0	1584.0	1859.0	154.0	2816.0	3564.0	3971.0	5324.0	6875.0
15	1178.6	1697.1	1991.8	154.0	3017.1	3818.6	4254.6	5704.3	7366.1
16	1257.1	1810.3	2124.6	154.0	3218.3	4073.1	4538.3	6084.6	7857.1
17	1335.7	1923.4	2257.4	154.0	3419.4	4327.7	4821.9	6464.9	8348.2
18	1414.3	2036.6	2390.1	154.0	3620.6	4582.3	5105.6	6845.1	8839.3
19	1492.9	2149.7	2522.9	154.0	3821.7	4836.9	5389.2	7225.4	9330.4
20	1571.4	2262.9	2655.7	154.0	4022.9	5091.4	5672.9	7605.7	9821.4
21	1650.0	2376.0	2788.5	154.0	4224.0	5346.0	5956.5	7986.0	10312.5
22	1728.6	2489.1	2921.3	154.0	4425.1	5600.6	6240.1	8366.3	10803.6
23	1807.1	2602.3	3054.1	154.0	4626.3	5855.1	6523.8	8746.6	11294.6
24	1885.7	2715.4	3186.9	154.0	4827.4	6109.7	6807.4	9126.9	11785.7
25	1964.3	2828.6	3319.6	154.0	5028.6	6364.3	7091.1	9507.1	12276.8

LAMPIRAN 7

COLUMN DESIGN CHART



Gambar L7.1 ***Column Design Chart***

LAMPIRAN 8

OUTPUT PROGRAM *CSiCol* v8.4.0

1. Metode A

```
General
Project Information

        Project         Tugas Akhir
        Job No
        Company
        Designer
        Remarks

        Software        CSICOL (Version: 8.4  (Rev. 0))
        File Name       E:\Rie  nebeng\Skripsi  Rie\TA  Riri\gambar\kolom\SNI  statik
        \c39

Working Units   Metric (m, Ton, Ton-m, kg/cm^2)
Design Code     ACI-318-05



Sections in Current File
Column:C39
Basic Design Parameters
        Caption                              = C39
        Default Concrete Strength, Fc        = 250 kg/cm^2
        Default Concrete Modulus, Ec         = 240000 kg/cm^2
        Maximum Concrete Strain              = 0.003 in/in

        Rebar Set                            = ASTM
        Default Rebar Yeild Strength, Fy     = 4000 kg/cm^2
        Default Rebar Modulus, Es            = 2000000 kg/cm^2
        Default Cover to Rebars              = 4.00 cm
        Maximum Steel Strain                 = Infinity

        Transverse Rebar Type   = Ties

        Total Shapes in Section        = 1
        Consider Slenderness   = No

Cross-section Shapes
        Shape   Width   Height  Conc Fc S/S Curve      Rebars
                cm      cm      kg/cm^2
        Rectangular Shape      70.00   70.00   250.00  ACI-Whitney        Rectangular
        12-#18

Rebar Properties
        Sr.No   Designation    Area    Cord-X   Cord-Y     Fy        S/S Curve
                               cm^2      cm       cm     kg/cm^2
        1       #18            25.8    4.00      4.00      4000      Elasto-Plastic
        2       #18            25.8    4.00     66.00      4000      Elasto-Plastic
        3       #18            25.8    66.00    66.00      4000      Elasto-Plastic
        4       #18            25.8    66.00     4.00      4000      Elasto-Plastic
        5       #18            25.8    4.00     24.67      4000      Elasto-Plastic
        6       #18            25.8    4.00     45.33      4000      Elasto-Plastic
        7       #18            25.8    24.67    66.00      4000      Elasto-Plastic
        8       #18            25.8    45.33    66.00      4000      Elasto-Plastic
        9       #18            25.8    66.00    45.33      4000      Elasto-Plastic
        10      #18            25.8    66.00    24.67      4000      Elasto-Plastic
        11      #18            25.8    45.33     4.00      4000      Elasto-Plastic
        12      #18            25.8    24.67     4.00      4000      Elasto-Plastic
```

```
        12-#18
        Total Area     = 309.8 cm^2
        Steel Ratio    = 6.32 %

Basic Section Properties:
        Total Width          = 70.00 cm
        Total Height         = 70.00 cm
        Center, Xo           =  0.00 cm
        Center, Yo           =  0.00 cm

        X-bar (Right)        = 35.00 cm
        X-bar (Left)         = 35.00 cm
        Y-bar (Top)          = 35.00 cm
        Y-bar (Bot)          = 35.00 cm

        Transformed Properties:
        Base Material = fc'  = 250 kg/cm^2
        Area, A              = 4,900.0 cm^2
        Inertia, I33         = 2.00E+06 cm^4
        Inertia, I22         = 2.00E+06 cm^4
        Inertia, I32         = 0.00E+00 cm^4

        Radius, r3           = 20.207 cm
        Radius, r2           = 20.207 cm

Additional Section Properties:
        Transformed Properties:
        Base Material = fc'  = 250 kg/cm^2
        Modulus, S3(Top)     = 5.72E+04 cm^3
        Modulus, S3(Bot)     = 5.72E+04 cm^3
        Modulus, S2(Left)    = 5.72E+04 cm^3
        Modulus, S2(Right)   = 5.72E+04 cm^3

        Plastic Modulus, Z3  = 1.46E+05 cm^3
        Plastic Modulus, Z2  = 1.46E+05 cm^3
        Torsional, J         = 3.47E+06 cm^4
        Shear Area, A3       = 4,256.7 cm^2
        Shear Area, A2       = 4,256.7 cm^2

        Principal Angle      = 0.00E+00 Deg
        Inertia, I33'        = 2.00E+06 cm^4
        Inertia, I22'        = 2.00E+06 cm^4


Framing Along-X
         Total C/C Length, Lc        = 3.500 m
         Unsupported Length, Lu      = 3.000 m
         Framing Type                = 4
         Framing Case                = 0
         K Factor, Braced            = 1.00
         Kl/r, Braced                = 14.85
         K Factor, Unbraced          = 1.00
         Kl/r, Unbraced              = 14.85

Framing Along-Y
         Total C/C Length, Lc        = 3.500 m
         Unsupported Length, Lu      = 3.000 m
         Framing Type                = 4
         Framing Case                = 0
         K Factor, Braced            = 1.00
         Kl/r, Braced                = 14.85
         K Factor, Unbraced          = 1.00
         Kl/r, Unbraced              = 14.85

Final Design Loads
```

Sr.No	Combination	Load Pu ton	Mux-Bot ton-m	Muy-Bot ton-m	Mux-Top ton-m	Muy-Top ton-m
1	comb1	272.95	-3.72	-0.36	6.18	0.65
2	comb2	304.95	-4.40	-0.33	7.34	0.68
3	comb3	266.94	35.97	11.57	-6.53	-2.80
4	comb4	261.76	35.62	-11.65	-6.52	3.76
5	comb5	245.34	-43.11	-12.22	18.41	3.99
6	comb6	250.53	-42.75	11.00	18.40	-2.57
7	comb7	267.25	8.83	38.46	2.18	-10.38

8	comb8	249.96	7.65	-38.93	2.21	11.50
9	comb9	245.03	-15.97	-39.10	9.69	11.57
10	comb10	262.33	-14.78	38.29	9.66	-10.31
11	comb11	186.27	37.15	11.66	-8.50	-2.98
12	comb12	181.08	36.79	-11.55	-8.49	3.59
13	comb13	164.66	-41.93	-12.12	16.45	3.81
14	comb14	169.85	-41.57	11.09	16.44	-2.75
15	comb15	186.58	10.01	38.55	0.22	-10.56
16	comb16	169.28	8.82	-38.84	0.25	11.33
17	comb17	164.36	-14.79	-39.01	7.73	11.39
18	comb18	181.65	-13.61	38.38	7.70	-10.49

Result Summary

Sr.No	Combination	Pu(ton)	Cap.Ratio-Bot	Cap.Ratio-Top	Remarks
1	comb1	272.95	0.237	0.237	Capacity OK
2	comb2	304.95	0.265	0.265	Capacity OK
3	comb3	266.94	0.292	0.232	Capacity OK
4	comb4	261.76	0.289	0.227	Capacity OK
5	comb5	245.34	0.30	0.221	Capacity OK
6	comb6	250.53	0.301	0.224	Capacity OK
7	comb7	267.25	0.296	0.232	Capacity OK
8	comb8	249.96	0.285	0.217	Capacity OK
9	comb9	245.03	0.296	0.213	Capacity OK
10	comb10	262.33	0.302	0.228	Capacity OK
11	comb11	186.27	0.247	0.162	Capacity OK
12	comb12	181.08	0.243	0.157	Capacity OK
13	comb13	164.66	0.251	0.16	Capacity OK
14	comb14	169.85	0.251	0.163	Capacity OK
15	comb15	186.58	0.249	0.162	Capacity OK
16	comb16	169.28	0.237	0.147	Capacity OK
17	comb17	164.36	0.247	0.153	Capacity OK
18	comb18	181.65	0.252	0.163	Capacity OK

2. Metode B

General
Project Information

Project Tugas Akhir
Job No
Company
Designer
Remarks

Software CSICOL (Version: 8.4 (Rev. 0))
File Name E:\Rie nebeng\Skripsi Rie\TA Riri\gambar\kolom\SNI statik \c39

Working Units Metric (m, Ton, Ton-m, kg/cm^2)
Design Code ACI-318-05

Sections in Current File
Column:C39
Basic Design Parameters

Caption = C39
Default Concrete Strength, Fc = 250 kg/cm^2
Default Concrete Modulus, Ec = 240000 kg/cm^2
Maximum Concrete Strain = 0.003 in/in

Rebar Set = ASTM
Default Rebar Yeild Strength, Fy = 4000 kg/cm^2
Default Rebar Modulus, Es = 2000000 kg/cm^2
Default Cover to Rebars = 4.00 cm
Maximum Steel Strain = Infinity

Transverse Rebar Type = Ties

Total Shapes in Section = 1
Consider Slenderness = No

Cross-section Shapes

Shape	Width cm	Height cm	Conc Fc kg/cm^2	S/S Curve	Rebars
Rectangular Shape	70.00	70.00	250.00	ACI-Whitney	Rectangular 12-#18

Rebar Properties

Sr.No	Designation	Area cm^2	Cord-X cm	Cord-Y cm	Fy kg/cm^2	S/S Curve
1	#18	25.8	4.00	4.00	4000	Elasto-Plastic
2	#18	25.8	4.00	66.00	4000	Elasto-Plastic
3	#18	25.8	66.00	66.00	4000	Elasto-Plastic
4	#18	25.8	66.00	4.00	4000	Elasto-Plastic
5	#18	25.8	4.00	24.67	4000	Elasto-Plastic
6	#18	25.8	4.00	45.33	4000	Elasto-Plastic
7	#18	25.8	24.67	66.00	4000	Elasto-Plastic
8	#18	25.8	45.33	66.00	4000	Elasto-Plastic
9	#18	25.8	66.00	45.33	4000	Elasto-Plastic
10	#18	25.8	66.00	24.67	4000	Elasto-Plastic
11	#18	25.8	45.33	4.00	4000	Elasto-Plastic
12	#18	25.8	24.67	4.00	4000	Elasto-Plastic

12-#18
Total Area = 309.8 cm^2
Steel Ratio = 6.32 %

Basic Section Properties:
Total Width = 70.00 cm
Total Height = 70.00 cm
Center, Xo = 0.00 cm
Center, Yo = 0.00 cm

X-bar (Right) = 35.00 cm
X-bar (Left) = 35.00 cm
Y-bar (Top) = 35.00 cm
Y-bar (Bot) = 35.00 cm

Transformed Properties:
Base Material = fc' = 250 kg/cm^2
Area, A = 4,900.0 cm^2
Inertia, I33 = 2.00E+06 cm^4
Inertia, I22 = 2.00E+06 cm^4
Inertia, I32 = 0.00E+00 cm^4

Radius, r3 = 20.207 cm
Radius, r2 = 20.207 cm

Additional Section Properties:
Transformed Properties:
Base Material = fc' = 250 kg/cm^2
Modulus, S3(Top) = 5.72E+04 cm^3
Modulus, S3(Bot) = 5.72E+04 cm^3
Modulus, S2(Left) = 5.72E+04 cm^3
Modulus, S2(Right) = 5.72E+04 cm^3

Plastic Modulus, Z3 = 1.46E+05 cm^3
Plastic Modulus, Z2 = 1.46E+05 cm^3
Torsional, J = 3.47E+06 cm^4
Shear Area, A3 = 4,256.7 cm^2
Shear Area, A2 = 4,256.7 cm^2

Principal Angle = 0.00E+00 Deg
Inertia, I33' = 2.00E+06 cm^4
Inertia, I22' = 2.00E+06 cm^4

Framing Along-X
Total C/C Length, Lc = 3.500 m
Unsupported Length, Lu = 3.000 m
Framing Type = 4
Framing Case = 0
K Factor, Braced = 1.00
Kl/r, Braced = 14.85
K Factor, Unbraced = 1.00

Kl/r, Unbraced = 14.85

Framing Along-Y

Total C/C Length, Lc	= 3.500 m
Unsupported Length, Lu	= 3.000 m
Framing Type	= 4
Framing Case	= 0
K Factor, Braced	= 1.00
Kl/r, Braced	= 14.85
K Factor, Unbraced	= 1.00
Kl/r, Unbraced	= 14.85

Final Design Loads

Sr.No	Combination	Load Pu	Mux-Bot	Muy-Bot	Mux-Top	Muy-Top
		ton	ton-m	ton-m	ton-m	ton-m
1	comb1	272.95	-3.72	-0.36	6.18	0.65
2	comb2	304.95	-4.40	-0.36	7.34	0.68
3	comb3	259.08	7.13	2.94	2.56	-0.34
4	comb4	257.65	7.03	-3.44	2.57	1.47
5	comb5	253.21	-14.27	-3.59	9.31	1.53
6	comb6	254.63	-14.17	2.79	9.31	-0.28
7	comb7	259.19	-0.21	10.34	4.92	-2.42
8	comb8	254.43	-0.54	-10.94	4.93	3.59
9	comb9	253.10	-6.93	-10.98	6.95	3.61
10	comb10	257.85	-6.60	10.29	6.95	-2.41
11	comb11	178.40	8.31	3.04	0.60	-0.52
12	comb12	176.97	8.21	-3.34	0.60	1.29
13	comb13	172.53	-13.09	-3.50	7.35	1.35
14	comb14	173.96	-12.99	2.88	7.35	-0.45
15	comb15	178.51	0.97	10.43	2.96	-2.60
16	comb16	173.76	0.64	-10.84	2.97	3.42
17	comb17	172.42	-5.75	-10.89	4.99	3.43
18	comb18	177.18	-5.42	10.38	4.98	-2.58

Result Summary

Sr.No	Combination	Pu(ton)	Cap.Ratio-Bot	Cap.Ratio-Top	Remarks
1	comb1	272.95	0.237	0.237	Capacity OK
2	comb2	304.95	0.265	0.265	Capacity OK
3	comb3	259.08	0.225	0.225	Capacity OK
4	comb4	257.65	0.224	0.224	Capacity OK
5	comb5	253.21	0.22	0.22	Capacity OK
6	comb6	254.63	0.221	0.221	Capacity OK
7	comb7	259.19	0.225	0.225	Capacity OK
8	comb8	254.43	0.221	0.221	Capacity OK
9	comb9	253.10	0.22	0.22	Capacity OK
10	comb10	257.85	0.224	0.224	Capacity OK
11	comb11	178.40	0.155	0.155	Capacity OK
12	comb12	176.97	0.154	0.154	Capacity OK
13	comb13	172.53	0.158	0.15	Capacity OK
14	comb14	173.96	0.158	0.151	Capacity OK
15	comb15	178.51	0.155	0.155	Capacity OK
16	comb16	173.76	0.151	0.151	Capacity OK
17	comb17	172.42	0.155	0.15	Capacity OK
18	comb18	177.18	0.157	0.154	Capacity OK

3. Metode C

General
Project Information

Project	Tugas Akhir
Job No	
Company	
Designer	
Remarks	

Software	CSICOL (Version: 8.4 (Rev. 0))
File Name	E:\Rie nebeng\Skripsi Rie\TA Riri\gambar\kolom\SNI statik \c39

Working Units Metric (m, Ton, Ton-m, kg/cm^2)
Design Code ACI-318-05

Sections in Current File
Column:C39
Basic Design Parameters

Caption	= C39
Default Concrete Strength, Fc	= 250 kg/cm^2
Default Concrete Modulus, Ec	= 240000 kg/cm^2
Maximum Concrete Strain	= 0.003 in/in
Rebar Set	= ASTM
Default Rebar Yeild Strength, Fy	= 4000 kg/cm^2
Default Rebar Modulus, Es	= 2000000 kg/cm^2
Default Cover to Rebars	= 4.00 cm
Maximum Steel Strain	= Infinity
Transverse Rebar Type	= Ties
Total Shapes in Section	= 1
Consider Slenderness	= No

Cross-section Shapes

Shape	Width cm	Height cm	Conc Fc kg/cm^2	S/S Curve	Rebars
Rectangular Shape	70.00	70.00	250.00	ACI-Whitney	Rectangular 12-#18

Rebar Properties

Sr.No	Designation	Area cm^2	Cord-X cm	Cord-Y cm	Fy kg/cm^2	S/S Curve
1	#18	25.8	4.00	4.00	4000	Elasto-Plastic
2	#18	25.8	4.00	66.00	4000	Elasto-Plastic
3	#18	25.8	66.00	66.00	4000	Elasto-Plastic
4	#18	25.8	66.00	4.00	4000	Elasto-Plastic
5	#18	25.8	4.00	24.67	4000	Elasto-Plastic
6	#18	25.8	4.00	45.33	4000	Elasto-Plastic
7	#18	25.8	24.67	66.00	4000	Elasto-Plastic
8	#18	25.8	45.33	66.00	4000	Elasto-Plastic
9	#18	25.8	66.00	45.33	4000	Elasto-Plastic
10	#18	25.8	66.00	24.67	4000	Elasto-Plastic
11	#18	25.8	45.33	4.00	4000	Elasto-Plastic
12	#18	25.8	24.67	4.00	4000	Elasto-Plastic

12-#18
Total Area = 309.8 cm^2
Steel Ratio = 6.32 %

Basic Section Properties:

Total Width	= 70.00 cm
Total Height	= 70.00 cm
Center, Xo	= 0.00 cm
Center, Yo	= 0.00 cm
X-bar (Right)	= 35.00 cm
X-bar (Left)	= 35.00 cm
Y-bar (Top)	= 35.00 cm
Y-bar (Bot)	= 35.00 cm

Transformed Properties:

Base Material = fc'	= 250 kg/cm^2
Area, A	= 4,900.0 cm^2
Inertia, I33	= 2.00E+06 cm^4
Inertia, I22	= 2.00E+06 cm^4
Inertia, I32	= 0.00E+00 cm^4
Radius, r3	= 20.207 cm
Radius, r2	= 20.207 cm

Additional Section Properties:
Transformed Properties:

Base Material = fc'	= 250 kg/cm^2
Modulus, S3(Top)	= 5.72E+04 cm^3
Modulus, S3(Bot)	= 5.72E+04 cm^3
Modulus, S2(Left)	= 5.72E+04 cm^3
Modulus, S2(Right)	= 5.72E+04 cm^3

```
Plastic Modulus, Z3   = 1.46E+05 cm^3
Plastic Modulus, Z2   = 1.46E+05 cm^3
Torsional, J          = 3.47E+06 cm^4
Shear Area, A3        = 4,256.7 cm^2
Shear Area, A2        = 4,256.7 cm^2

Principal Angle       = 0.00E+00 Deg
Inertia, I33'         = 2.00E+06 cm^4
Inertia, I22'         = 2.00E+06 cm^4
```

```
Framing Along-X
         Total C/C Length, Lc        = 3.500 m
         Unsupported Length, Lu      = 3.000 m
         Framing Type                = 4
         Framing Case                = 0
         K Factor, Braced            = 1.00
         Kl/r, Braced                = 14.85
         K Factor, Unbraced          = 1.00
         Kl/r, Unbraced              = 14.85

Framing Along-Y
         Total C/C Length, Lc        = 3.500 m
         Unsupported Length, Lu      = 3.000 m
         Framing Type                = 4
         Framing Case                = 0
         K Factor, Braced            = 1.00
         Kl/r, Braced                = 14.85
         K Factor, Unbraced          = 1.00
         Kl/r, Unbraced              = 14.85
```

Final Design Loads

Sr.No	Combination	Load Pu ton	Mux-Bot ton-m	Muy-Bot ton-m	Mux-Top ton-m	Muy-Top ton-m
1	comb1	288.24	-3.72	-0.36	6.19	0.65
2	comb2	318.06	-4.41	-0.36	7.35	0.68
3	comb3	278.78	-34.03	-29.83	15.58	9.00
4	comb4	194.83	-32.86	-29.73	13.61	8.83

Result Summary

Sr.No	Combination	Pu(ton)	Cap.Ratio-Bot	Cap.Ratio-Top	Remarks
1	comb1	288.24	0.25	0.25	Capacity OK
2	comb2	318.06	0.276	0.276	Capacity OK
3	comb3	278.78	0.332	0.244	Capacity OK
4	comb4	194.83	0.28	0.181	Capacity OK

4. Metode D

```
General
Project Information

       Project        Tugas Akhir
       Job No
       Company
       Designer
       Remarks

       Software       CSICOL (Version: 8.4  (Rev. 0))
       File Name      E:\Rie  nebeng\Skripsi  Rie\TA  Riri\gambar\kolom\SNI  statik
       \c39

Working Units   Metric (m, Ton, Ton-m, kg/cm^2)
Design Code     ACI-318-05


Sections in Current File
Column:C39
Basic Design Parameters
        Caption                            = C39
        Default Concrete Strength, Fc      = 250 kg/cm^2
        Default Concrete Modulus, Ec       = 240000 kg/cm^2
```

```
         Maximum Concrete Strain             = 0.003 in/in

         Rebar Set                           = ASTM
         Default Rebar Yeild Strength, Fy    = 4000 kg/cm^2
         Default Rebar Modulus, Es           = 2000000 kg/cm^2
         Default Cover to Rebars             = 4.00 cm
         Maximum Steel Strain                = Infinity

         Transverse Rebar Type  = Ties

         Total Shapes in Section       = 1
          Consider Slenderness  = No

Cross-section Shapes
        Shape   Width   Height  Conc Fc S/S Curve      Rebars
                cm      cm      kg/cm^2
        Rectangular Shape      70.00   70.00   250.00  ACI-Whitney        Rectangular
        12-#18

Rebar Properties
        Sr.No   Designation    Area    Cord-X   Cord-Y     Fy       S/S Curve
                               cm^2     cm       cm      kg/cm^2
        1       #18            25.8     4.00      4.00     4000     Elasto-Plastic
        2       #18            25.8     4.00     66.00     4000     Elasto-Plastic
        3       #18            25.8    66.00     66.00     4000     Elasto-Plastic
        4       #18            25.8    66.00      4.00     4000     Elasto-Plastic
        5       #18            25.8     4.00     24.67     4000     Elasto-Plastic
        6       #18            25.8     4.00     45.33     4000     Elasto-Plastic
        7       #18            25.8    24.67     66.00     4000     Elasto-Plastic
        8       #18            25.8    45.33     66.00     4000     Elasto-Plastic
        9       #18            25.8    66.00     45.33     4000     Elasto-Plastic
        10      #18            25.8    66.00     24.67     4000     Elasto-Plastic
        11      #18            25.8    45.33      4.00     4000     Elasto-Plastic
        12      #18            25.8    24.67      4.00     4000     Elasto-Plastic

                12-#18
                Total Area     = 309.8 cm^2
                Steel Ratio    = 6.32 %

Basic Section Properties:
         Total Width           = 70.00 cm
         Total Height          = 70.00 cm
         Center, Xo            =  0.00 cm
         Center, Yo            =  0.00 cm

         X-bar (Right)         = 35.00 cm
         X-bar (Left)          = 35.00 cm
         Y-bar (Top)           = 35.00 cm
         Y-bar (Bot)           = 35.00 cm

         Transformed Properties:
         Base Material = fc'   = 250 kg/cm^2
         Area, A               = 4,900.0 cm^2
         Inertia, I33          = 2.00E+06 cm^4
         Inertia, I22          = 2.00E+06 cm^4
         Inertia, I32          = 0.00E+00 cm^4

         Radius, r3            = 20.207 cm
         Radius, r2            = 20.207 cm

Additional Section Properties:
         Transformed Properties:
         Base Material = fc'   = 250 kg/cm^2
         Modulus, S3(Top)      = 5.72E+04 cm^3
         Modulus, S3(Bot)      = 5.72E+04 cm^3
         Modulus, S2(Left)     = 5.72E+04 cm^3
         Modulus, S2(Right)    = 5.72E+04 cm^3

         Plastic Modulus, Z3   = 1.46E+05 cm^3
         Plastic Modulus, Z2   = 1.46E+05 cm^3
         Torsional, J          = 3.47E+06 cm^4
         Shear Area, A3        = 4,256.7 cm^2
         Shear Area, A2        = 4,256.7 cm^2
```

```
Principal Angle       = 0.00E+00 Deg
Inertia, I33'         = 2.00E+06 cm^4
Inertia, I22'         = 2.00E+06 cm^4
```

Framing Along-X

```
Total C/C Length, Lc        = 3.500 m
Unsupported Length, Lu      = 3.000 m
Framing Type                = 4
Framing Case                = 0
K Factor, Braced            = 1.00
Kl/r, Braced                = 14.85
K Factor, Unbraced          = 1.00
Kl/r, Unbraced              = 14.85
```

Framing Along-Y

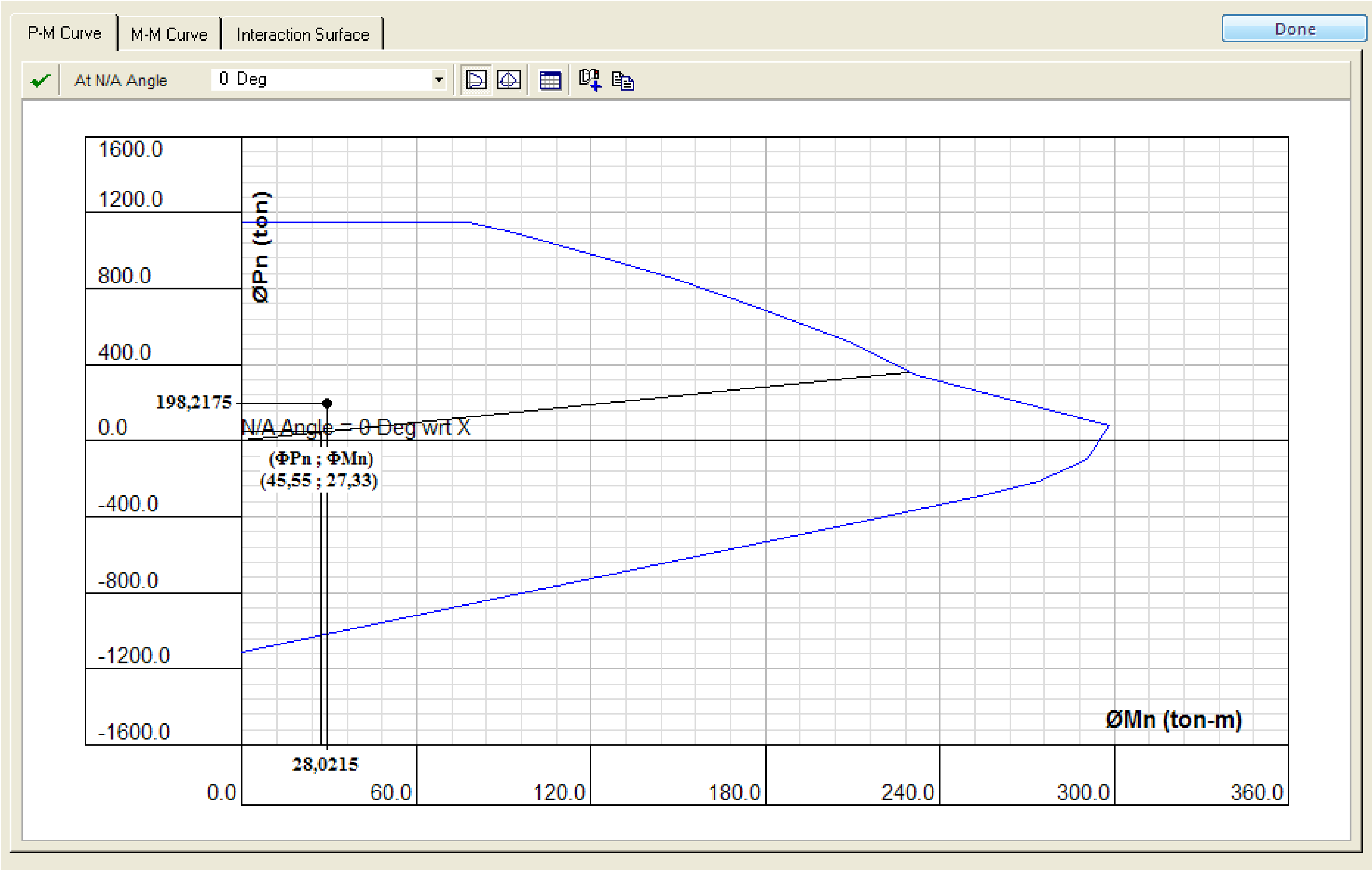
```
Total C/C Length, Lc        = 3.50  m
Unsupported Length, Lu      = 3.000 m
Framing Type                = 4
Framing Case                = 0
K Factor, Braced            = 1.00
Kl/r, Braced                = 14.85
K Factor, Unbraced          = 1.00
Kl/r, Unbraced              = 14.85
```

Final Design Loads

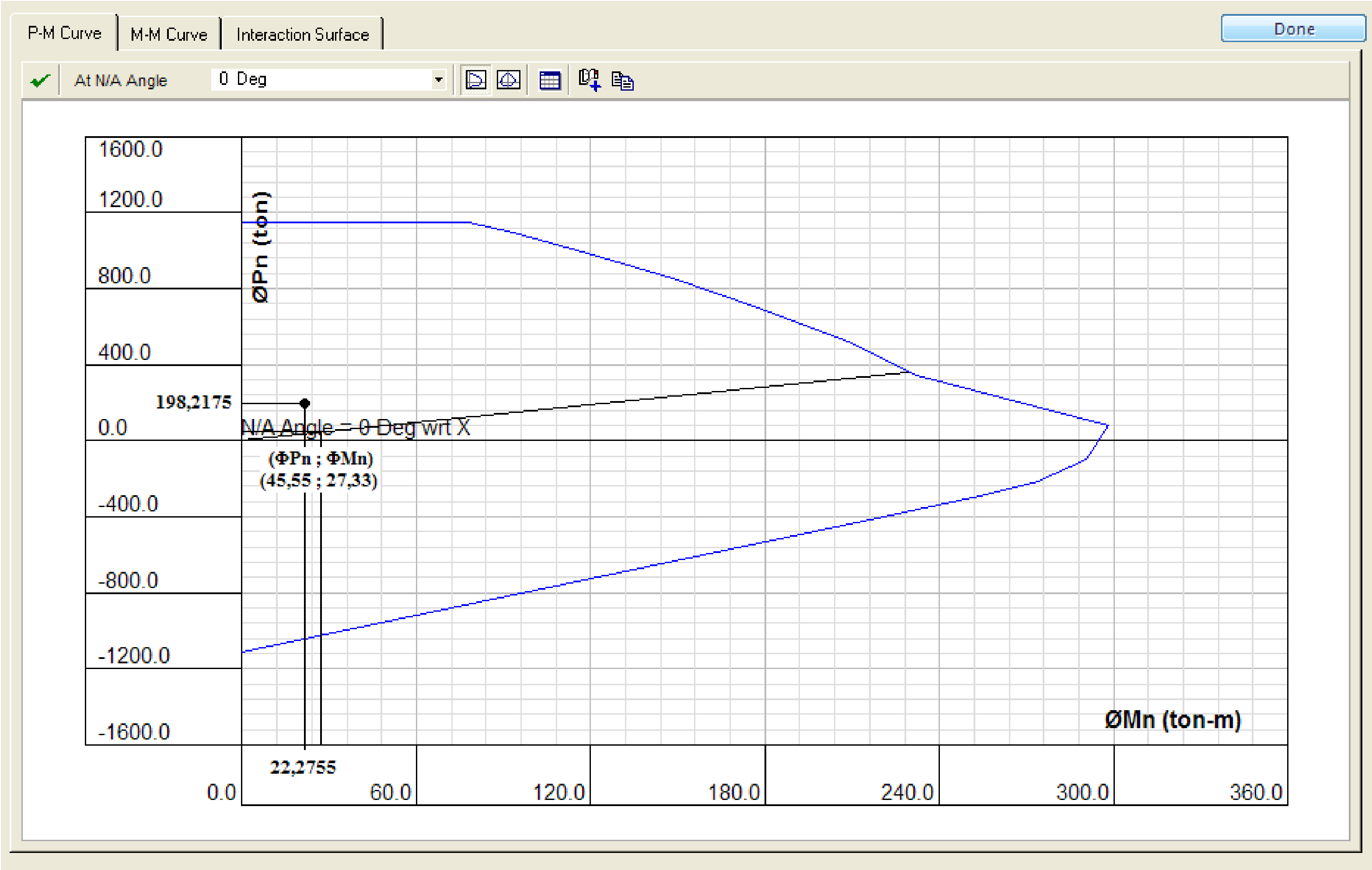
Sr.No	Combination	Load Pu ton	Mux-Bot ton-m	Muy-Bot ton-m	Mux-Top ton-m	Muy-Top ton-m
1	Comb1	272.95	-3.72	-0.36	6.18	0.65
2	comb2	304.95	-4.40	-0.36	7.34	0.65
3	Comb3	265.67	-34.03	-29.83	15.57	9.00
4	Comb4	185.00	-32.85	-29.73	13.62	8.82

Result Summary

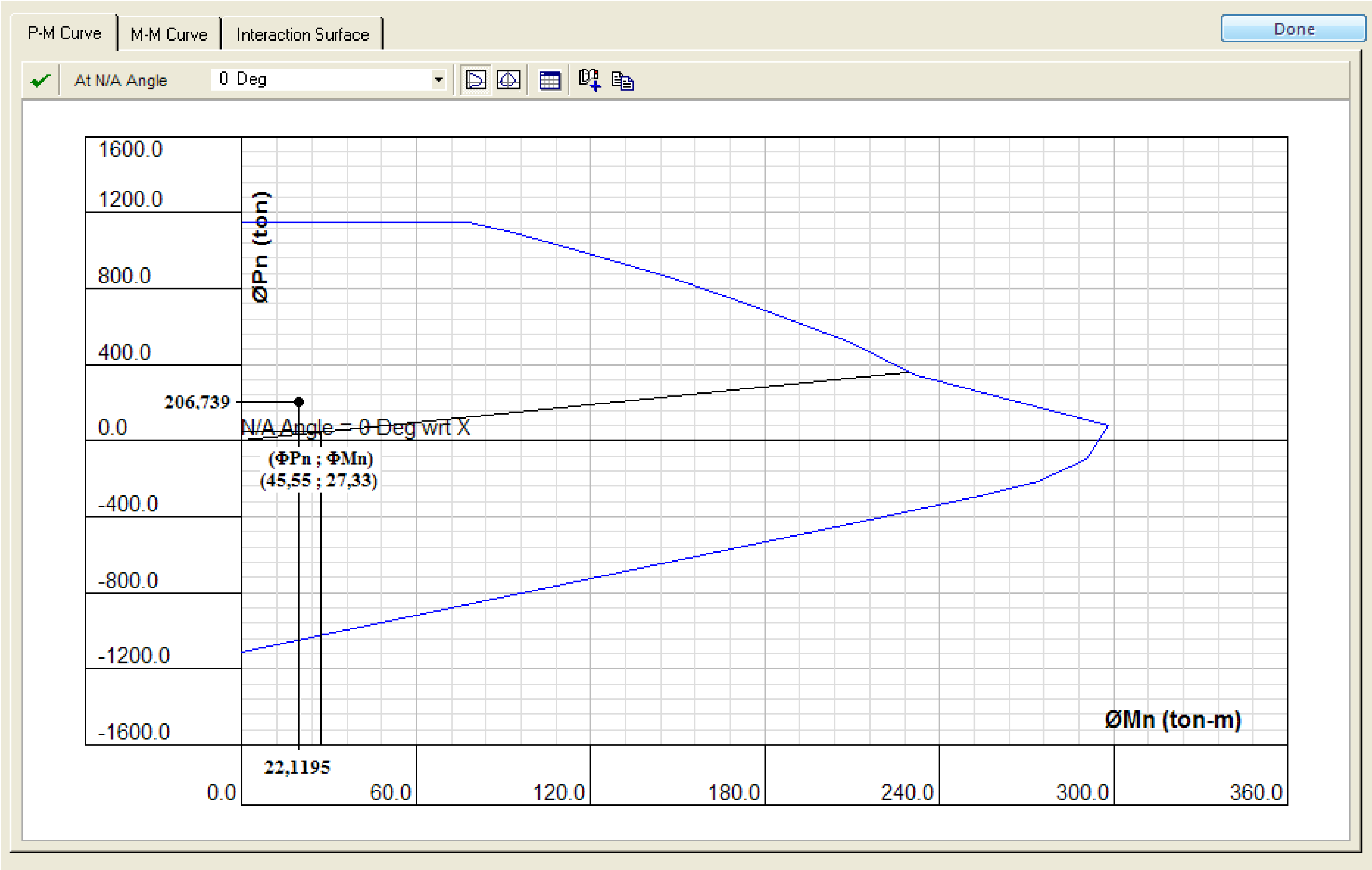
Sr.No	Combination	Pu(ton)	Cap.Ratio-Bot	Cap.Ratio-Top	Remarks
1	Comb1	272.95	0.237	0.237	Capacity OK
2	comb2	304.95	0.265	0.265	Capacity OK
3	Comb3	265.67	0.325	0.235	Capacity OK
4	Comb4	185.00	0.275	0.174	Capacity OK



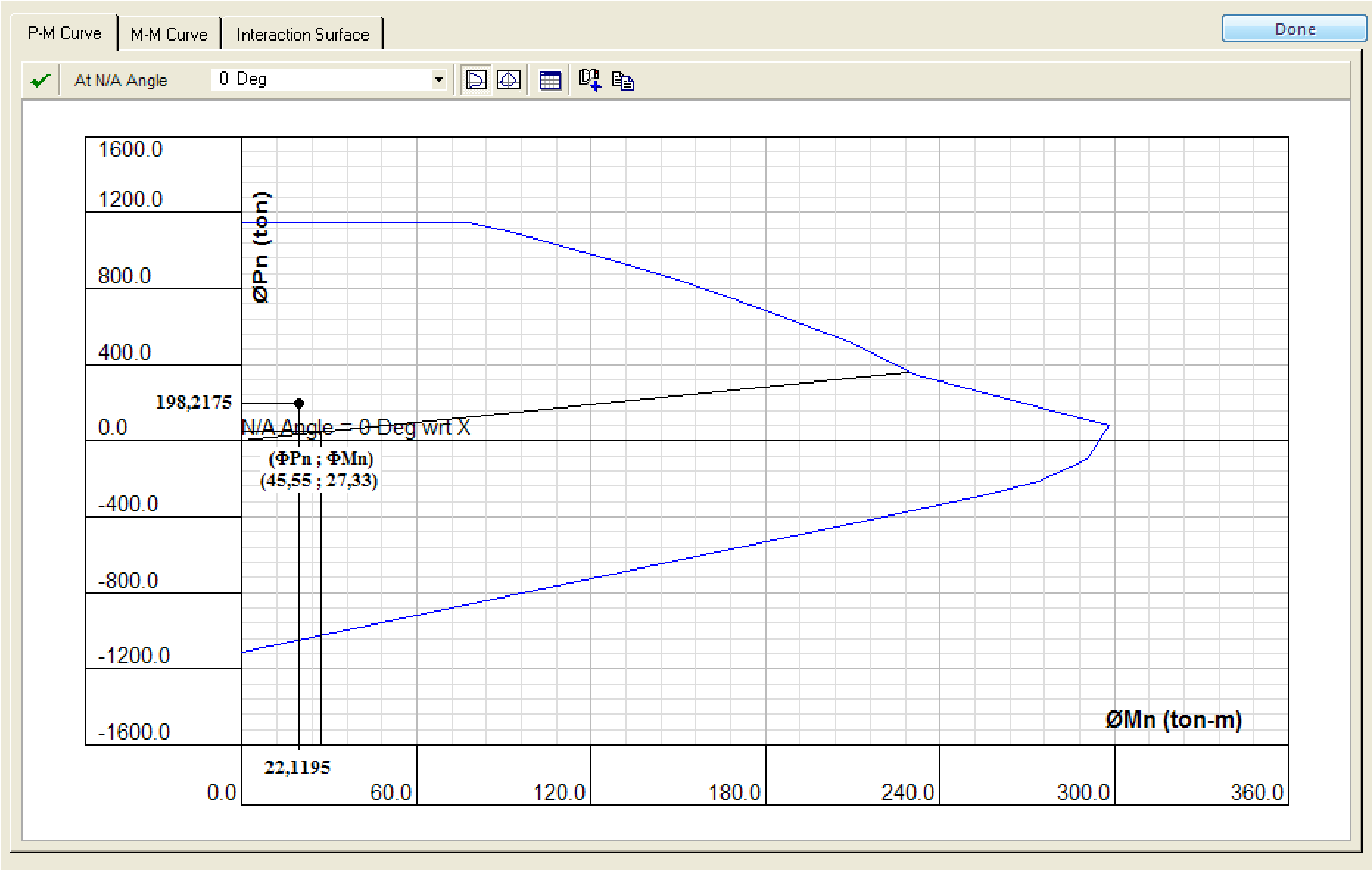
Gambar L8.1 Kurva Hubungan φP_n dengan φM_n Metode A



Gambar L8.2 Kurva Hubungan φP_n dengan φM_n Metode B



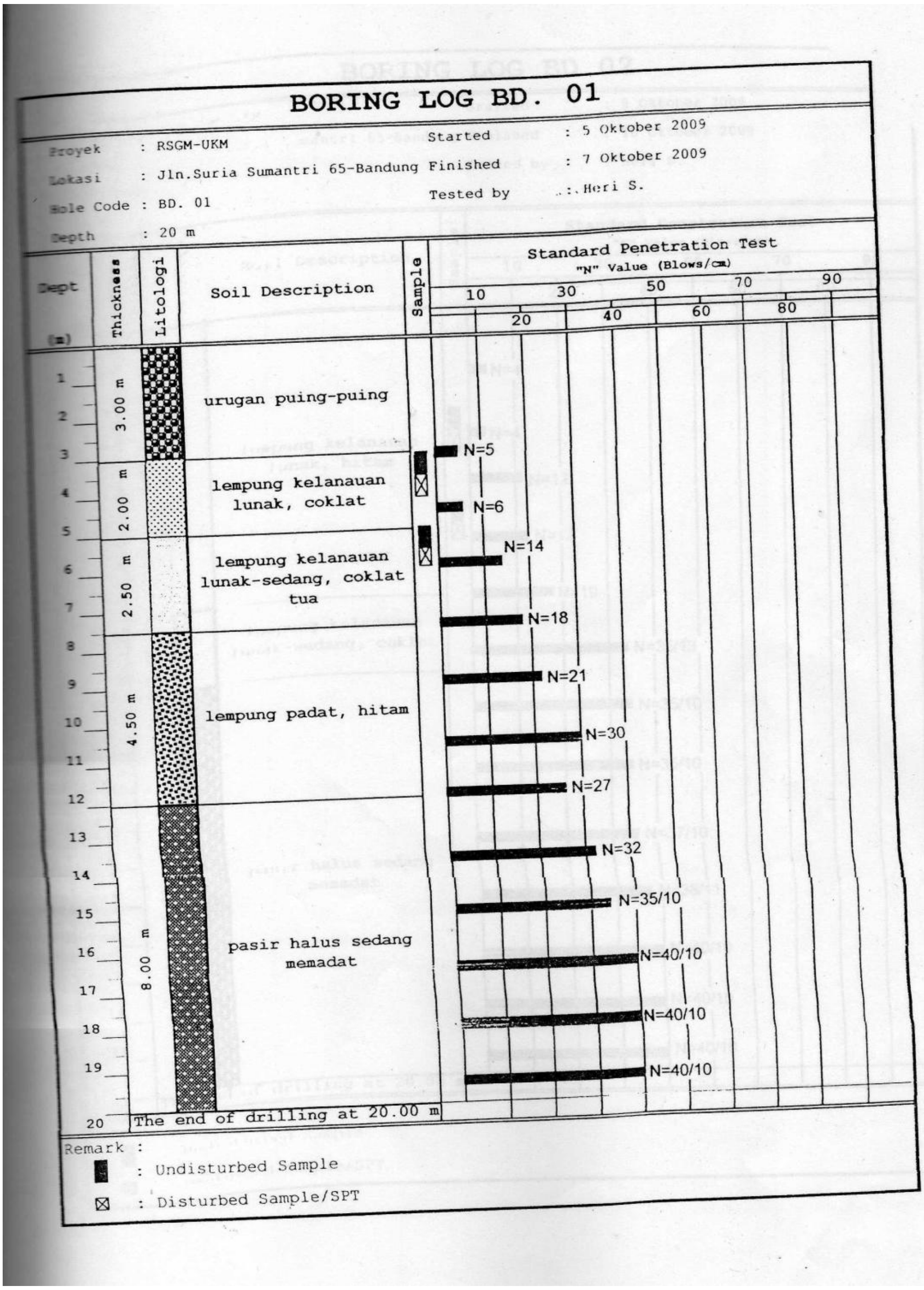
Gambar L8.3 Kurva Hubungan φP_n dengan φM_n Metode C



Gambar L8.4 Kurva Hubungan φP_n dengan φM_n Metode D

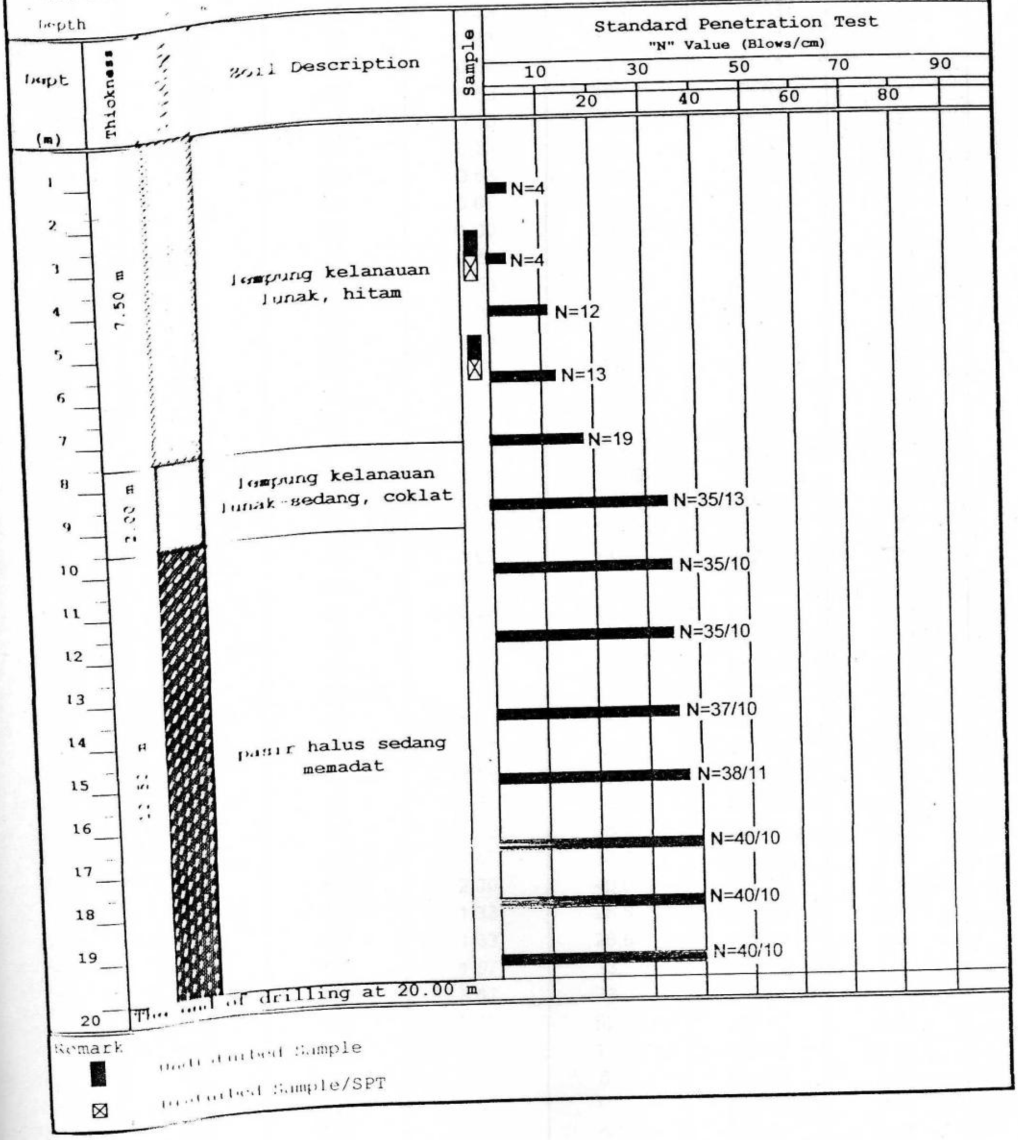
LAMPIRAN 9

DATA SONDIR



BORING LOG BD.02

Proyek	:mantri 65-Bandung	Started	: 8 Oktober 2009
Lokasi		Finished	: 10 Oktober 2009
Hole Code		Tested by	: Heri S.
Depth			



DATA SONDIR PADA PEMBANGUNAN RUMAH SAKIT GIGI & MULUT
UNIVERSITAS KRISTEN MARANATHA
JALAN PROF. DRG. SURIA SUMANTRI 65 BANDUNG

Test Point : S-1 (Depan kanan poliklinik)
Elevation : ± 0.00 m dari Elevasi muka tanah setempat

Form No. : II/2
Date : 10 Oktober 2009

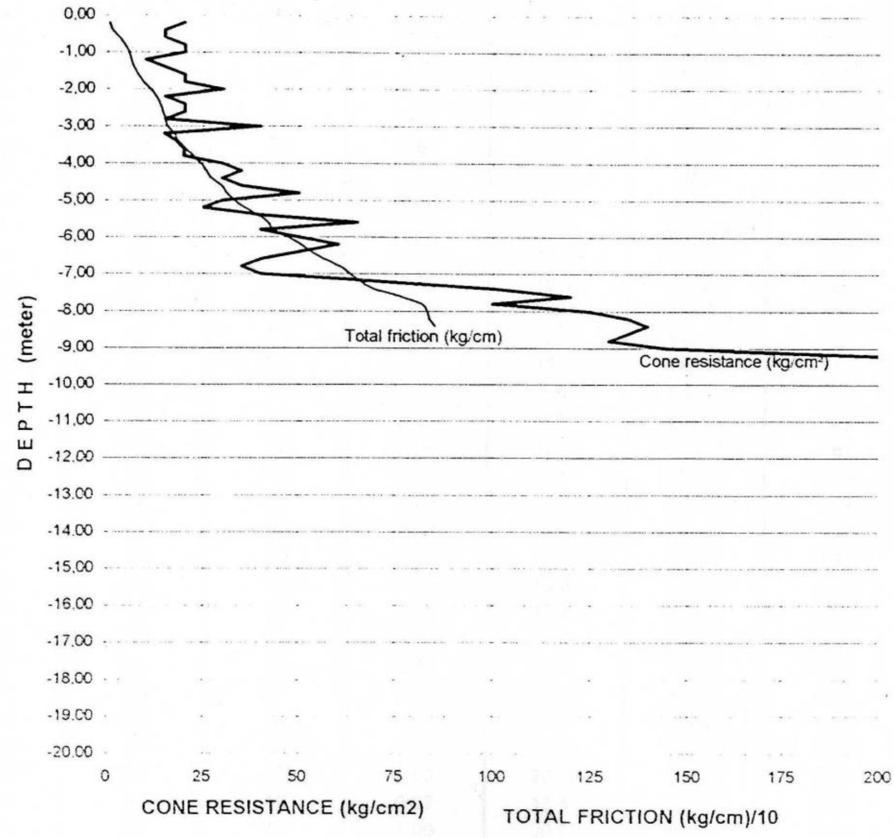
Depth (m)	R1 = qc (kgf/sqcm)	R2 (kgf/sqcm)	fs (kgf/sqcm)	LF (kgf/cm)	TF (kgf/cm)	fs/qc (%)
-0,20	20	25	0,33	6,67	6,67	1,67
-0,40	15	20	0,33	6,67	13,33	2,22
-0,60	15	28	0,87	17,33	30,67	5,78
-0,80	20	30	0,67	13,33	44,00	3,33
-1,00	20	28	0,53	10,67	54,67	2,67
-1,20	10	15	0,33	6,67	61,33	3,33
-1,40	15	20	0,33	6,67	68,00	2,22
-1,60	20	28	0,53	10,67	78,67	2,67
-1,80	20	30	0,67	13,33	92,00	3,33
-2,00	30	45	1,00	20,00	112,00	3,33
-2,20	15	25	0,67	13,33	125,33	4,44
-2,40	20	28	0,53	10,67	136,00	2,67
-2,60	20	25	0,33	6,67	142,67	1,67
-2,80	15	20	0,33	6,67	149,33	2,22
-3,00	40	45	0,33	6,67	156,00	0,83
-3,20	15	30	1,00	20,00	176,00	6,67
-3,40	18	25	0,47	9,33	185,33	2,59
-3,60	20	35	1,00	20,00	205,33	5,00
-3,80	20	35	1,00	20,00	225,33	5,00
-4,00	30	45	1,00	20,00	245,33	3,33
-4,20	35	45	0,67	13,33	258,67	1,90
-4,40	30	40	0,67	13,33	272,00	2,22
-4,60	35	55	1,33	26,67	298,67	3,81
-4,80	50	60	0,67	13,33	312,00	1,33
-5,00	30	45	1,00	20,00	332,00	3,33
-5,20	25	45	1,33	26,67	358,67	5,33
-5,40	40	65	1,67	33,33	392,00	4,17
-5,60	65	85	1,33	26,67	418,67	2,05
-5,80 √	40	50	0,67	13,33	432,00	1,67
-6,00	50	80	2,00	40,00	472,00	4,00
-6,20	60	85	1,67	33,33	505,33	2,78
-6,40	50	70	1,33	26,67	532,00	2,67
-6,60	40	65	1,67	33,33	565,33	4,17
-6,80	35	65	2,00	40,00	605,33	5,71
-7,00	40	60	1,33	26,67	632,00	3,33
-7,20	70	90	1,33	26,67	658,67	1,90
-7,40	100	125	1,67	33,33	692,00	1,67
-7,60	120	175	3,67	73,33	765,33	3,06
-7,80	100	140	2,67	53,33	818,67	2,67
-8,00	125	135	0,67	13,33	832,00	0,53
-8,20	135	140	0,33	6,67	838,67	0,25
-8,40	140	150	0,67	13,33	852,00	0,48
-8,60	135	140	0,33	6,67	858,67	0,25
-8,80	130	150	1,33	26,67	885,33	1,03
-9,00	145	165	1,33	26,67	912,00	0,92
-9,20	200					

DATA SONDIR PADA PEMBANGUNAN RUMAH SAKIT GIGI & MULUT
UNIVERSITAS KRISTEN MARANATHA
JALAN PROF. DRG. SURIA SUMANTRI 65 BANDUNG

Test Point : S-1 (Depan kanan poliklinik)
Elevation : ± 0.00 m dari Elevasi muka tanah setempat

Form No. : II/2
D a t e : 10 Oktober 2(

CONE RESISTANCE vs DEPTH
&
TOTAL FRICTION vs DEPTH



DATA SONDIR PADA PEMBANGUNAN RUMAH SAKIT GIGI & MULUT
UNIVERSITAS KRISTEN MARANATHA
JALAN PROF. DRG. SURIA SUMANTRI 65 BANDUNG

Test Point : S-2 (Belakang kanan poliklinik)
Elevation : ± 0.00 m dari Elevasi muka tanah setempat

Form No. : II/2
Date : 10 Oktober 2009

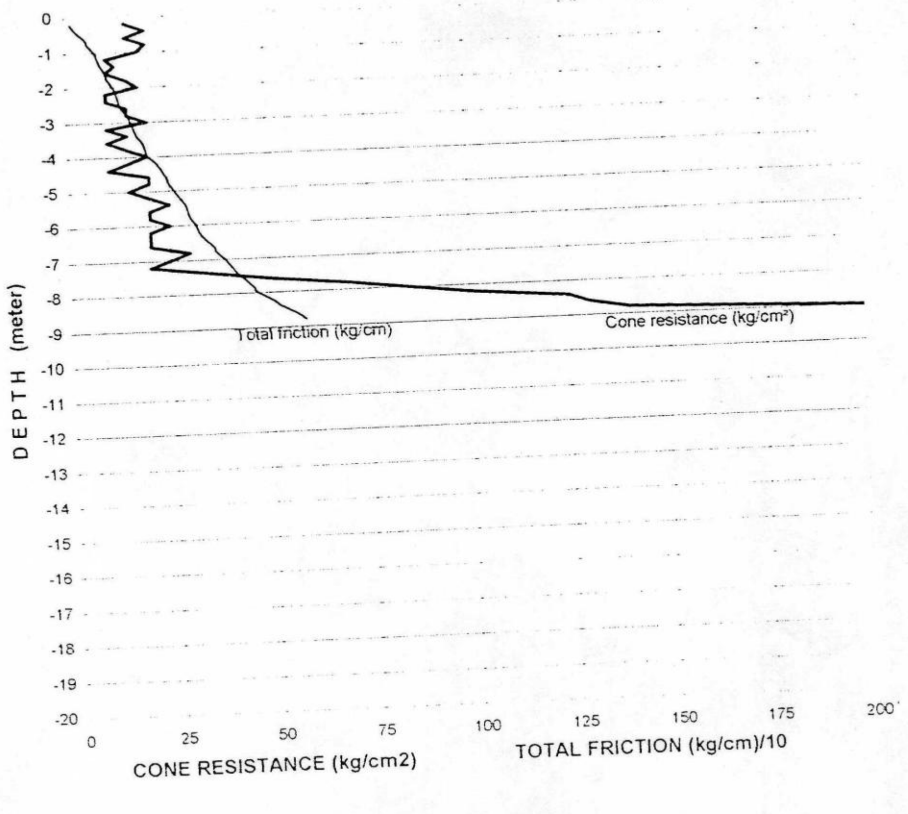
Depth (m)	R1 = qc (kgf/sqcm)	R2 (kgf/sqcm)	fs (kgf/sqcm)	LF (kgf/cm)	TF (kgf/cm)	fs/qc (%)
-0,20	15	25	0,67	13,33	13,33	4,44
-0,40	20	30	0,67	13,33	26,67	3,33
-0,60	15	30	1,00	20,00	46,67	6,67
-0,80	20	28	0,53	10,67	57,33	2,67
-1,00	18	30	0,80	16,00	73,33	4,44
-1,20	10	15	0,33	6,67	80,00	3,33
-1,40	12	18	0,40	8,00	88,00	3,33
-1,60	10	18	0,53	10,67	98,67	5,33
-1,80	15	20	0,33	6,67	105,33	2,22
-2,00	18	28	0,67	13,33	118,67	3,70
-2,20	10	15	0,33	6,67	125,33	3,33
-2,40	10	14	0,27	5,33	130,67	2,67
-2,60	15	20	0,33	6,67	137,33	2,22
-2,80	15	25	0,67	13,33	150,67	4,44
-3,00	20	28	0,53	10,67	161,33	2,67
-3,20	10	15	0,33	6,67	168,00	3,33
-3,40	15	20	0,33	6,67	174,67	2,22
-3,60	10	20	0,67	13,33	188,00	6,67
-3,80	15	20	0,33	6,67	194,67	2,22
-4,00	20	25	0,33	6,67	201,33	1,67
-4,20	15	30	1,00	20,00	221,33	6,67
-4,40	10	18	0,53	10,67	232,00	5,33
-4,60	20	28	0,53	10,67	242,67	2,67
-4,80	20	25	0,33	6,67	249,33	1,67
-5,00	15	25	0,67	13,33	262,67	4,44
-5,20	20	30	0,67	13,33	276,00	3,33
-5,40	25	35	0,67	13,33	289,33	2,67
-5,60	20	25	0,33	6,67	296,00	1,67
-5,80	20	25	0,33	6,67	302,67	1,67
-6,00	25	35	0,67	13,33	316,00	2,67
-6,20	20	25	0,33	6,67	322,67	1,67
-6,40	20	30	0,67	13,33	336,00	3,33
-6,60	20	35	1,00	20,00	356,00	5,00
-6,80	30	40	0,67	13,33	369,33	2,22
-7,00	25	40	1,00	20,00	389,33	4,00
-7,20	20	30	0,67	13,33	402,67	3,33
-7,40	35	45	0,67	13,33	416,00	1,90
-7,60	50	65	1,00	20,00	436,00	2,00
-7,80	70	85	1,00	20,00	456,00	1,43
-8,00	85	95	0,67	13,33	469,33	0,78
-8,20	100	125	1,67	33,33	502,67	1,67
-8,40	125	145	1,33	26,67	529,33	1,07
-8,60	130	155	1,67	33,33	562,67	1,28
-8,80	140	160	1,33	26,67	589,33	0,95
-9,00	200					

DATA SONDIR PADA PEMBANGUNAN RUMAH SAKIT GIGI & MULUT
UNIVERSITAS KRISTEN MARANATHA
JALAN PROF. DRG. SURIA SUMANTRI 65 BANDUNG

Form No. : II/2
Date : 10 Oktober 2009

Test Point : S-2 (Belakang kanan poliklinik)
Elevation : ± 0.00 m dari Elevasi
muka tanah setempat

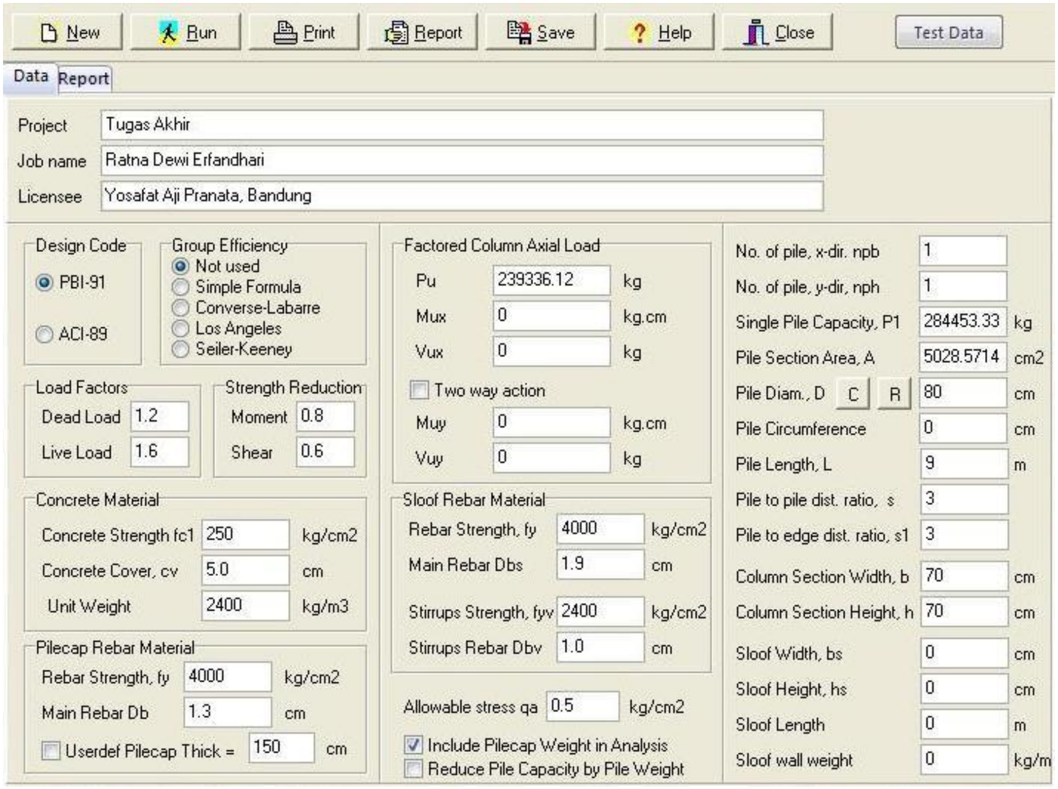
CONE RESISTANCE vs DEPTH
&
TOTAL FRICTION vs DEPTH



LAMPIRAN 10

OUTPUT PROGRAM *CONCRETE PILECAP DESIGN*

1. Metode A



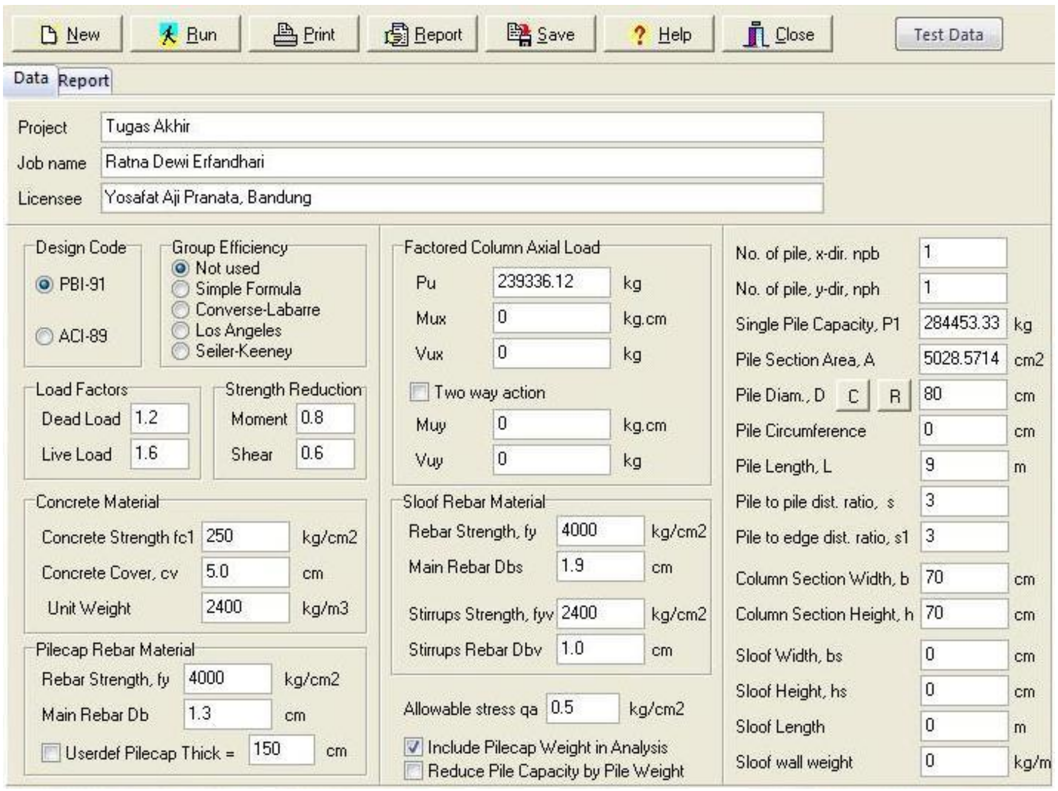
Gambar L10.1 Tampilan Program *Concrete Pilecap Design*

```
PILECAP - Pilecap Design V.1.1
(C) Nathan Madutujuh, 1999-2003
Engineering Software Research Center

  Project   : Tugas Akhir
  Job Name  : Ratna Dewi Erfandhari


PILECAP DESIGN

  Design Code      : PBI-91

    Factor for Dead Load          = 1.20
    Factor for Live Load          = 1.60
    Strength Reduction for Moment = 0.80
    Strength Reduction for Shear  = 0.60

  Concrete Unit Weight,     Gm  =  2400.00 kg/m3
  Concrete Compr. Strength, fc1 =   250.00 kg/cm2
  Concrete Cover,           cv  =     5.00 cm
```

```
  Pilecap Rebar Yield Strength, fy  =  4000.00 kg/cm2
  Pilecap Rebar Diameter,       db  =     1.30 cm

  Sloof Rebar Yield Strength,   fys   =  4000.00 kg/cm2
  Sloof Stirrups Yield Strength, fy   =  2400.00 kg/cm2
  Sloof Main Rebar Diameter,     dbs  =     1.90 cm
  Sloof Stirrups Rebar Diameter, dbsv =     1.00 cm

  Allowable Soil Stress,    qa  =     0.50 kg/cm2

Unfactored Axial Load    P  =    170954.37 kg
Single Pile Capacity     P1 =    284453.33 kg
Single Pile Section Area A1       = 5028.57 cm2
Pile Length              L1       =    9.00 m
Pile Length Inside Pilecap L2     =   7.500 m
Pile Diameter            dp       =   80.00 cm
Pile to Pile Dist. Ratio  s       =    3.00 D
Pile to Edge Dist. Ratio s1       =    3.00 D

Column Section Width   b   =   70.00 cm
Column Section Height  h   =   70.00 cm

Sloof Section Width   b   =    0.00 cm
Sloof Section Height  h   =    0.00 cm

  Factored Axial Load,      Pu  =    239336.12 kg
  Factored Moment,         Mux  =         0.00 kg.cm
  Factored Shear,          Vux  =         0.00 kg.cm

Load Factor (Averaged) = 1.40

PILE DESIGN:

  Pile to Pile Distance    ds     =   240.00 cm
  Pile to Edge Distance   ds1     =   240.00 cm
  Number of Pile           np     = 1
  Weight of One Pile       W1     =     0.00 kg
  Single Pile Capacity  P1-W1     = 284453.33 kg
  Unfactored load, 1 Pile  P3     = 170954.37 kg
  Weight of All Piles      Wp     =     0.00 kg
  Weight of Pile Cap       Wc     = 30412.80 kg
  Pilecap Width            bp     =   480.00 cm
  Pilecap Length           hp     =   480.00 cm
  Pilecap Thickness        tp     =    55.00 cm (Included L2)

  Group Efficiency Method         = Not Applied
  Group Efficiency        eff     = 1.000
  Total Pile Capacity    Pcap     =    254040.53 kg

     Pcap > P   ----> OK

Shear Stress Checking:

   Beta Factor = h/b >= 1.0     =    1.00
   Punch Shear Force        Pp  =    170954.37 kg  (Unfactored)
   Punch Shear Force        Ppu =    239336.12 kg  (Factored)
   Critical Perimeter       Ko  =     500.0000 cm
   Punch Shear Stress       vc  =       9.8290 kg/cm2

Maximum shear stress (Without Phi factor)

   Punch Shear Capacity     vc1 =   16.67 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    8.33 kg/cm2
   Nett  Shear Capacity  vc max =   16.67 kg/cm2
   Nett  Shear Average       vc =    8.33 kg/cm2

Maximum shear stress (With Phi factor = 0.6)

   Punch Shear Capacity     vc1 =   10.00 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    5.00 kg/cm2
   Nett  Shear Capacity  vc max =   10.00 kg/cm2
   Nett  Shear Average       vc =    5.00 kg/cm2
```

```
Pilecap Thickness at Column Face:

   Punch Shear,          tp =   54.90 cm
   Nett  Shear, X-dir,   tp =   13.80 cm   ( 0 piles)
   Nett  Shear, Y-dir,   tp =   13.80 cm   ( 0 piles)

Pilecap Thickness at Edge:

   Nett  Shear, X-dir,   tp =    0.00 cm   ( 0 piles)
   Nett  Shear, Y-dir,   tp =    0.00 cm   ( 0 piles)

   Selected Pilecap Thickness tp =   55.00 cm (Included L2)

Pilecap Rebar Design:

  fc1 =  250.0 kg/cm2    Tp =  55.0 cm    db    =  1.3 cm
  fy  = 4000.0 kg/cm2    cv =   5.0 cm    romin = 0.00150

1. Bending Moment at Column Face, X-direction (0 piles)

     Not Applicable!

2. Bending Moment at Column Face, Y-direction (0 piles)

     Not Applicable!
```

2. Metode B



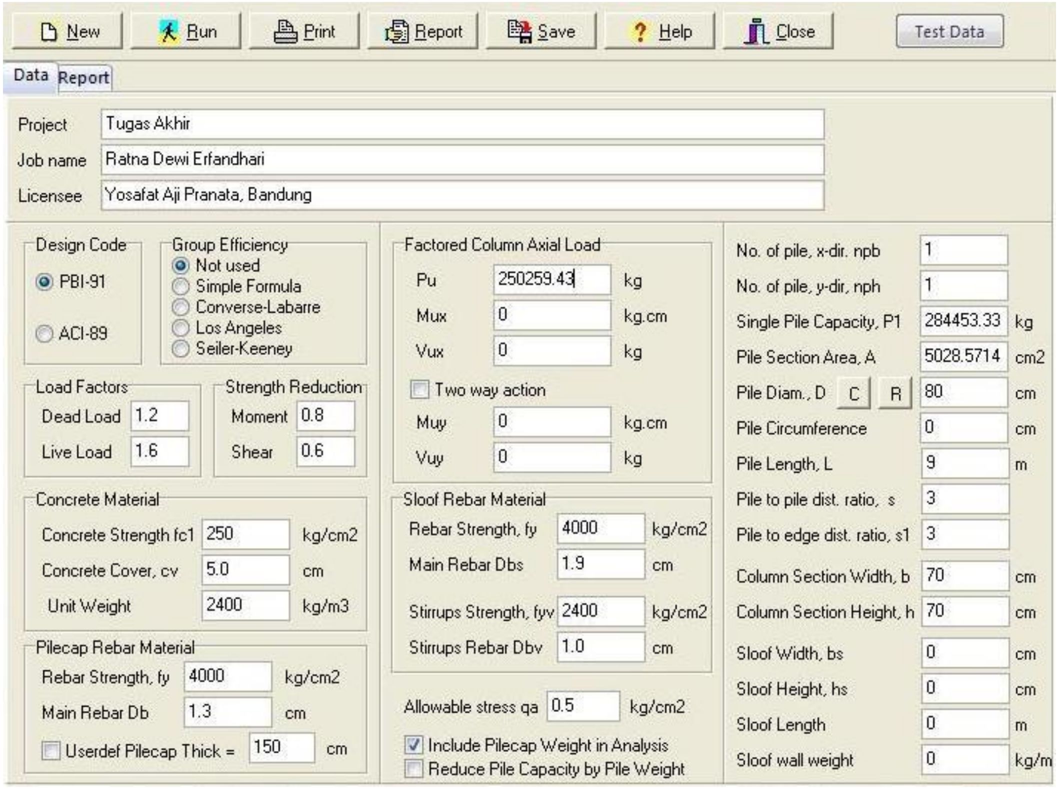
Gambar L10.2 Tampilan Program *Concrete Pilecap Design*

```
PILECAP - Pilecap Design V.1.1
(C) Nathan Madutujuh, 1999-2003
Engineering Software Research Center
```

```
  Project   : Tugas Akhir
  Job Name  : Ratna Dewi Erfandhari


PILECAP DESIGN

  Design Code     : PBI-91

    Factor for Dead Load          = 1.20
    Factor for Live Load          = 1.60
    Strength Reduction for Moment = 0.80
    Strength Reduction for Shear  = 0.60

  Concrete Unit Weight,     Gm  =  2400.00 kg/m3
  Concrete Compr. Strength, fc1 =   250.00 kg/cm2
  Concrete Cover,           cv  =     5.00 cm

  Pilecap Rebar Yield Strength, fy  =  4000.00 kg/cm2
  Pilecap Rebar Diameter,       db  =     1.30 cm

  Sloof Rebar Yield Strength,   fys   =  4000.00 kg/cm2
  Sloof Stirrups Yield Strength, fy   =  2400.00 kg/cm2
  Sloof Main Rebar Diameter,     dbs  =     1.90 cm
  Sloof Stirrups Rebar Diameter, dbsv =     1.00 cm

  Allowable Soil Stress,    qa  =     0.50 kg/cm2

Unfactored Axial Load    P  =    170954.37 kg
Single Pile Capacity     P1 =    284453.33 kg
Single Pile Section Area A1       = 5028.57 cm2
Pile Length              L1       =    9.00 m
Pile Length Inside Pilecap L2     =   7.500 m
Pile Diameter            dp       =   80.00 cm
Pile to Pile Dist. Ratio  s       =    3.00 D
Pile to Edge Dist. Ratio s1       =    3.00 D

Column Section Width   b   =   70.00 cm
Column Section Height  h   =   70.00 cm

Sloof Section Width   b    =    0.00 cm
Sloof Section Height  h    =    0.00 cm

  Factored Axial Load,      Pu  =    239336.12 kg
  Factored Moment,         Mux  =         0.00 kg.cm
  Factored Shear,          Vux  =         0.00 kg.cm

Load Factor (Averaged) = 1.40

PILE DESIGN:

  Pile to Pile Distance    ds     =   240.00 cm
  Pile to Edge Distance   ds1     =   240.00 cm
  Number of Pile           np     = 1
  Weight of One Pile       W1     =     0.00 kg
  Single Pile Capacity  P1-W1     = 284453.33 kg
  Unfactored load, 1 Pile  P3     = 170954.37 kg
  Weight of All Piles      Wp     =     0.00 kg
  Weight of Pile Cap       Wc     = 30412.80 kg
  Pilecap Width            bp     =   480.00 cm
  Pilecap Length           hp     =   480.00 cm
  Pilecap Thickness        tp     =    55.00 cm (Included L2)

  Group Efficiency Method         = Not Applied
  Group Efficiency        eff     = 1.000
  Total Pile Capacity    Pcap     =    254040.53 kg

     Pcap > P   ----> OK

Shear Stress Checking:

   Beta Factor = h/b >= 1.0     =    1.00
   Punch Shear Force        Pp  =    170954.37 kg  (Unfactored)
   Punch Shear Force        Ppu =    239336.12 kg  (Factored)
   Critical Perimeter       Ko  =     500.0000 cm
```

```
   Punch Shear Stress       vc  =       9.8290 kg/cm2

Maximum shear stress (Without Phi factor)

   Punch Shear Capacity     vc1 =   16.67 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    8.33 kg/cm2
   Nett  Shear Capacity  vc max =   16.67 kg/cm2
   Nett  Shear Average       vc =    8.33 kg/cm2

Maximum shear stress (With Phi factor = 0.6)

   Punch Shear Capacity     vc1 =   10.00 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    5.00 kg/cm2
   Nett  Shear Capacity  vc max =   10.00 kg/cm2
   Nett  Shear Average       vc =    5.00 kg/cm2

Pilecap Thickness at Column Face:

   Punch Shear,         tp =   54.90 cm
   Nett  Shear, X-dir,  tp =   13.80 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =   13.80 cm  ( 0 piles)

Pilecap Thickness at Edge:

   Nett  Shear, X-dir,  tp =    0.00 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =    0.00 cm  ( 0 piles)

   Selected Pilecap Thickness tp =   55.00 cm (Included L2)

Pilecap Rebar Design:

  fc1 =  250.0 kg/cm2   Tp =  55.0 cm   db    =  1.3 cm
  fy  = 4000.0 kg/cm2   cv =   5.0 cm   romin = 0.00150

1. Bending Moment at Column Face, X-direction (0 piles)

     Not Applicable!

2. Bending Moment at Column Face, Y-direction (0 piles)

     Not Applicable!
```

3. Metode C



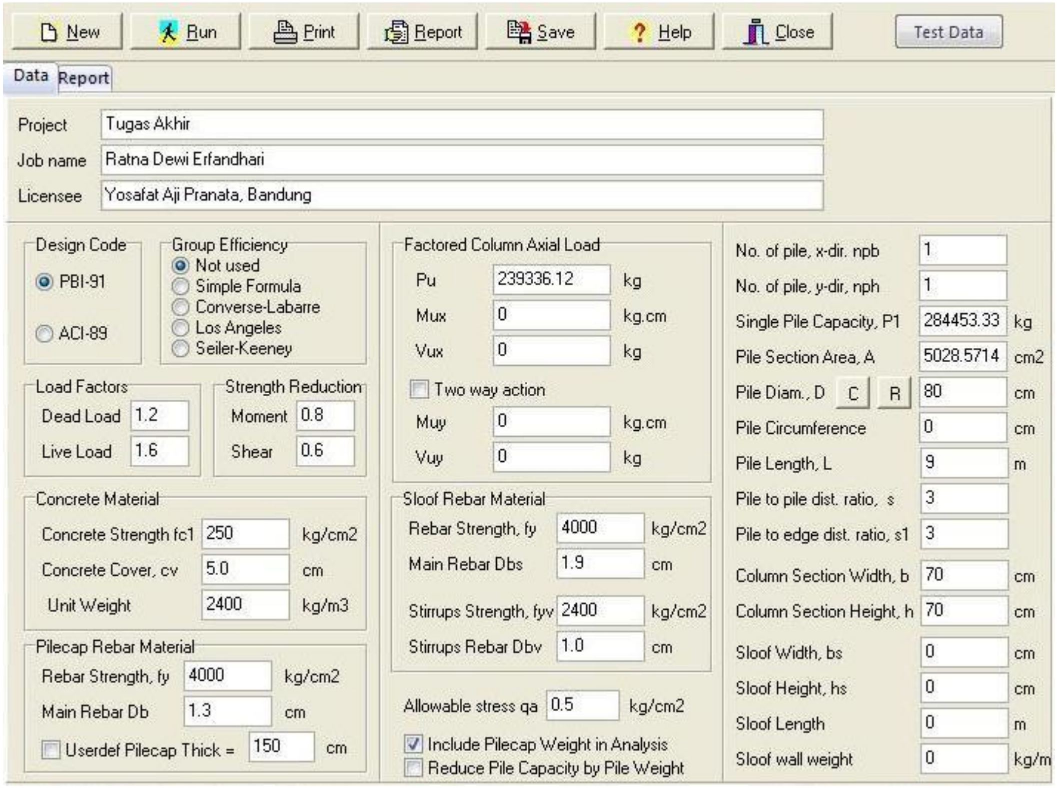
Gambar L10.3 Tampilan Program *Concrete Pilecap Design*

```
PILECAP - Pilecap Design V.1.1
(C) Nathan Madutujuh, 1999-2003
Engineering Software Research Center

  Licensee  :
  Project   : Tugas Akhir
  Job Name  : Ratna Dewi Erfandhari


PILECAP DESIGN

  Design Code     : PBI-91

    Factor for Dead Load          = 1.20
    Factor for Live Load          = 1.60
    Strength Reduction for Moment = 0.80
    Strength Reduction for Shear  = 0.60

  Concrete Unit Weight,     Gm  =  2400.00 kg/m3
  Concrete Compr. Strength, fc1 =   250.00 kg/cm2
  Concrete Cover,           cv  =     5.00 cm

  Pilecap Rebar Yield Strength, fy  =  4000.00 kg/cm2
  Pilecap Rebar Diameter,       db  =     1.30 cm

  Sloof Rebar Yield Strength,    fys   =  4000.00 kg/cm2
  Sloof Stirrups Yield Strength, fy    =  2400.00 kg/cm2
  Sloof Main Rebar Diameter,     dbs   =     1.90 cm
  Sloof Stirrups Rebar Diameter, dbsv  =     1.00 cm

  Allowable Soil Stress,    qa  =     0.50 kg/cm2

Unfactored Axial Load    P  =    178756.74 kg
Single Pile Capacity     P1 =    284453.33 kg
```

```
Single Pile Section Area A1       = 5028.57 cm2
Pile Length              L1       =    9.00 m
Pile Length Inside Pilecap L2     =   0.000 m
Pile Diameter            dp       =   80.00 cm
Pile to Pile Dist. Ratio  s       =    3.00 D
Pile to Edge Dist. Ratio s1       =    3.00 D

Column Section Width   b =   70.00 cm
Column Section Height  h =   70.00 cm

Sloof Section Width   b =    0.00 cm
Sloof Section Height  h =    0.00 cm

  Factored Axial Load,      Pu  =    250259.43 kg
  Factored Moment,         Mux  =         0.00 kg.cm
  Factored Shear,          Vux  =         0.00 kg.cm

Load Factor (Averaged) = 1.40

PILE DESIGN:

  Pile to Pile Distance    ds     =   240.00 cm
  Pile to Edge Distance   ds1     =   240.00 cm
  Number of Pile           np     = 1
  Weight of One Pile       W1     =     0.00 kg
  Single Pile Capacity  P1-W1     = 284453.33 kg
  Unfactored load, 1 Pile  P3     = 178756.74 kg
  Weight of All Piles      Wp     =     0.00 kg
  Weight of Pile Cap       Wc     = 33177.60 kg
  Pilecap Width            bp     =   480.00 cm
  Pilecap Length           hp     =   480.00 cm
  Pilecap Thickness        tp     =    60.00 cm (Included L2)

  Group Efficiency Method         = Not Applied
  Group Efficiency        eff     = 1.000
  Total Pile Capacity    Pcap     =    251275.73 kg

     Pcap > P   ----> OK

Shear Stress Checking:

   Beta Factor = h/b >= 1.0     =    1.00
   Punch Shear Force        Pp  =    178756.74 kg  (Unfactored)
   Punch Shear Force        Ppu =    250259.43 kg  (Factored)
   Critical Perimeter       Ko  =     520.0000 cm
   Punch Shear Stress       vc  =       8.9622 kg/cm2

Maximum shear stress (Without Phi factor)

   Punch Shear Capacity     vc1 =   16.67 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    8.33 kg/cm2
   Nett  Shear Capacity  vc max =   16.67 kg/cm2
   Nett  Shear Average       vc =    8.33 kg/cm2

Maximum shear stress (With Phi factor = 0.6)

   Punch Shear Capacity     vc1 =   10.00 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    5.00 kg/cm2
   Nett  Shear Capacity  vc max =   10.00 kg/cm2
   Nett  Shear Average       vc =    5.00 kg/cm2

Pilecap Thickness at Column Face:

   Punch Shear,         tp =   56.50 cm
   Nett  Shear, X-dir,  tp =   13.80 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =   13.80 cm  ( 0 piles)

Pilecap Thickness at Edge:

   Nett  Shear, X-dir,  tp =    0.00 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =    0.00 cm  ( 0 piles)

   Selected Pilecap Thickness tp =   60.00 cm (Included L2)
```

```
Pilecap Rebar Design:

  fc1 =  250.0 kg/cm2   Tp =  60.0 cm   db    =  1.3 cm
  fy  = 4000.0 kg/cm2   cv =   5.0 cm   romin = 0.00150

1. Bending Moment at Column Face, X-direction (0 piles)

     Not Applicable!

2. Bending Moment at Column Face, Y-direction (0 piles)

     Not Applicable!
```

4. Metode D



Gambar L10.4 Tampilan Program *Concrete Pilecap Design*

```
PILECAP - Pilecap Design V.1.1
(C) Nathan Madutujuh, 1999-2003
Engineering Software Research Center

  Licensee  :
  Project   : Tugas Akhir
  Job Name  : Ratna Dewi Erfandhari


PILECAP DESIGN

  Design Code     : PBI-91

    Factor for Dead Load          = 1.20
    Factor for Live Load          = 1.60
    Strength Reduction for Moment = 0.80
    Strength Reduction for Shear  = 0.60
```

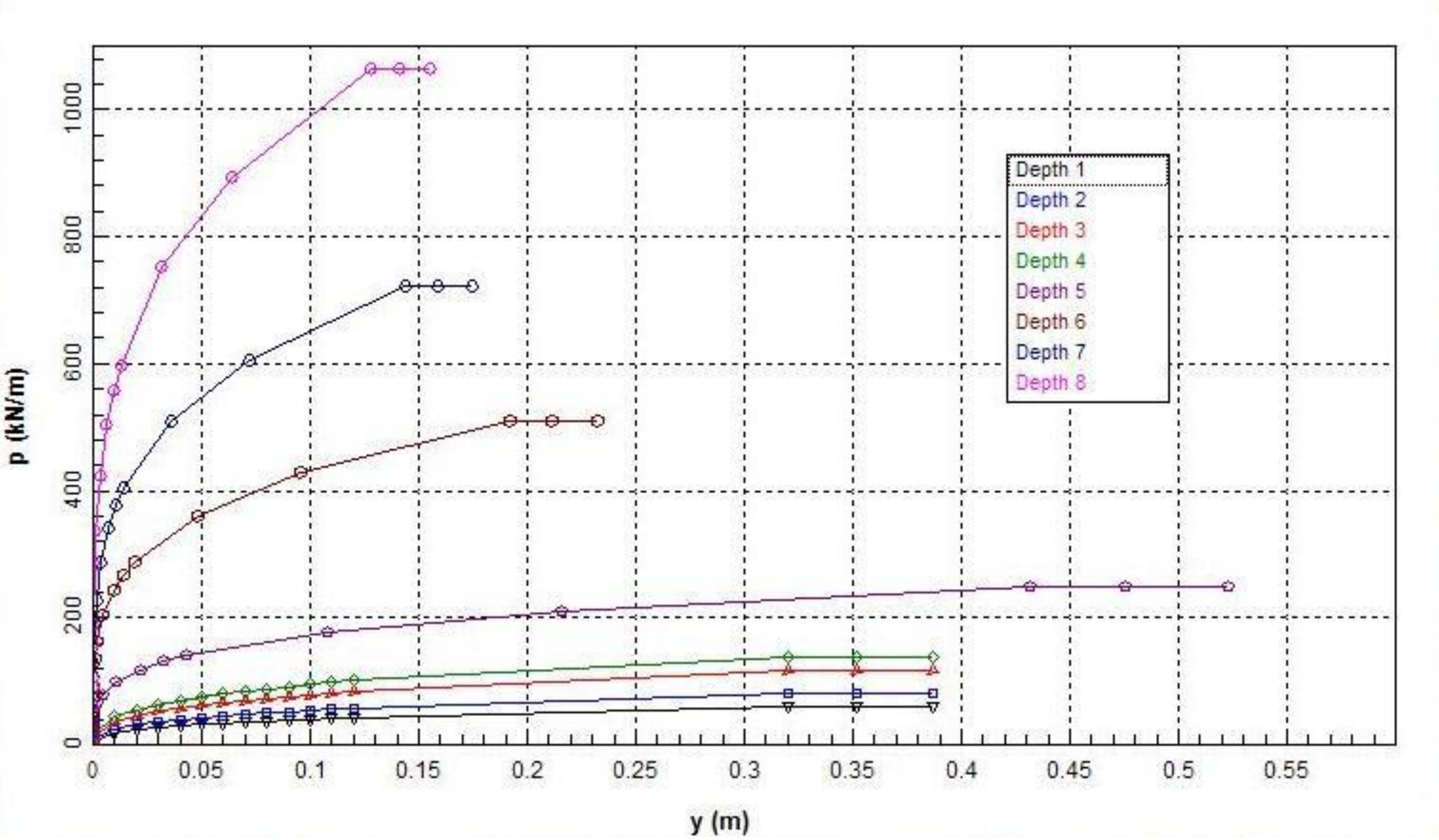
```
  Concrete Unit Weight,     Gm  =  2400.00 kg/m3
  Concrete Compr. Strength, fc1 =   250.00 kg/cm2
  Concrete Cover,           cv  =     5.00 cm

  Pilecap Rebar Yield Strength, fy  =  4000.00 kg/cm2
  Pilecap Rebar Diameter,       db  =     1.30 cm

  Sloof Rebar Yield Strength,   fys   =  4000.00 kg/cm2
  Sloof Stirrups Yield Strength, fy   =  2400.00 kg/cm2
  Sloof Main Rebar Diameter,     dbs  =     1.90 cm
  Sloof Stirrups Rebar Diameter, dbsv =     1.00 cm

  Allowable Soil Stress,    qa  =     0.50 kg/cm2

Unfactored Axial Load    P  =    170954.37 kg
Single Pile Capacity     P1 =    284453.33 kg
Single Pile Section Area A1       = 5028.57 cm2
Pile Length              L1       =    9.00 m
Pile Length Inside Pilecap L2     =   7.500 m
Pile Diameter            dp       =   80.00 cm
Pile to Pile Dist. Ratio  s       =    3.00 D
Pile to Edge Dist. Ratio s1       =    3.00 D

Column Section Width   b   =   70.00 cm
Column Section Height  h   =   70.00 cm

Sloof Section Width   b    =    0.00 cm
Sloof Section Height  h    =    0.00 cm

  Factored Axial Load,      Pu  =    239336.12 kg
  Factored Moment,         Mux  =         0.00 kg.cm
  Factored Shear,          Vux  =         0.00 kg.cm

Load Factor (Averaged) = 1.40

PILE DESIGN:

  Pile to Pile Distance    ds     =   240.00 cm
  Pile to Edge Distance   ds1     =   240.00 cm
  Number of Pile           np     = 1
  Weight of One Pile       W1     =     0.00 kg
  Single Pile Capacity  P1-W1     = 284453.33 kg
  Unfactored load, 1 Pile  P3     = 170954.37 kg
  Weight of All Piles      Wp     =     0.00 kg
  Weight of Pile Cap       Wc     = 30412.80 kg
  Pilecap Width            bp     =   480.00 cm
  Pilecap Length           hp     =   480.00 cm
  Pilecap Thickness        tp     =    55.00 cm (Included L2)

  Group Efficiency Method         = Not Applied
  Group Efficiency       eff      = 1.000
  Total Pile Capacity    Pcap     =    254040.53 kg

     Pcap > P   ----> OK

Shear Stress Checking:

   Beta Factor = h/b >= 1.0     =    1.00
   Punch Shear Force        Pp  =    170954.37 kg  (Unfactored)
   Punch Shear Force        Ppu =    239336.12 kg  (Factored)
   Critical Perimeter       Ko  =     500.0000 cm
   Punch Shear Stress       vc  =       9.8290 kg/cm2

Maximum shear stress (Without Phi factor)

   Punch Shear Capacity     vc1 =   16.67 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    8.33 kg/cm2
   Nett  Shear Capacity  vc max =   16.67 kg/cm2
   Nett  Shear Average       vc =    8.33 kg/cm2

Maximum shear stress (With Phi factor = 0.6)

   Punch Shear Capacity     vc1 =   10.00 kg/cm2  (Including Beta)
   Nett  Shear Capacity  vc min =    5.00 kg/cm2
```

```
   Nett  Shear Capacity  vc max =   10.00 kg/cm2
   Nett  Shear Average       vc =    5.00 kg/cm2

Pilecap Thickness at Column Face:

   Punch Shear,         tp =   54.90 cm
   Nett  Shear, X-dir,  tp =   13.80 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =   13.80 cm  ( 0 piles)

Pilecap Thickness at Edge:

   Nett  Shear, X-dir,  tp =    0.00 cm  ( 0 piles)
   Nett  Shear, Y-dir,  tp =    0.00 cm  ( 0 piles)

   Selected Pilecap Thickness tp =   55.00 cm (Included L2)

Pilecap Rebar Design:

  fc1 =  250.0 kg/cm2   Tp =  55.0 cm   db    =  1.3 cm
  fy  = 4000.0 kg/cm2   cv =   5.0 cm   romin = 0.00150

1. Bending Moment at Column Face, X-direction (0 piles)

     Not Applicable!

2. Bending Moment at Column Face, Y-direction (0 piles)

     Not Applicable!
```

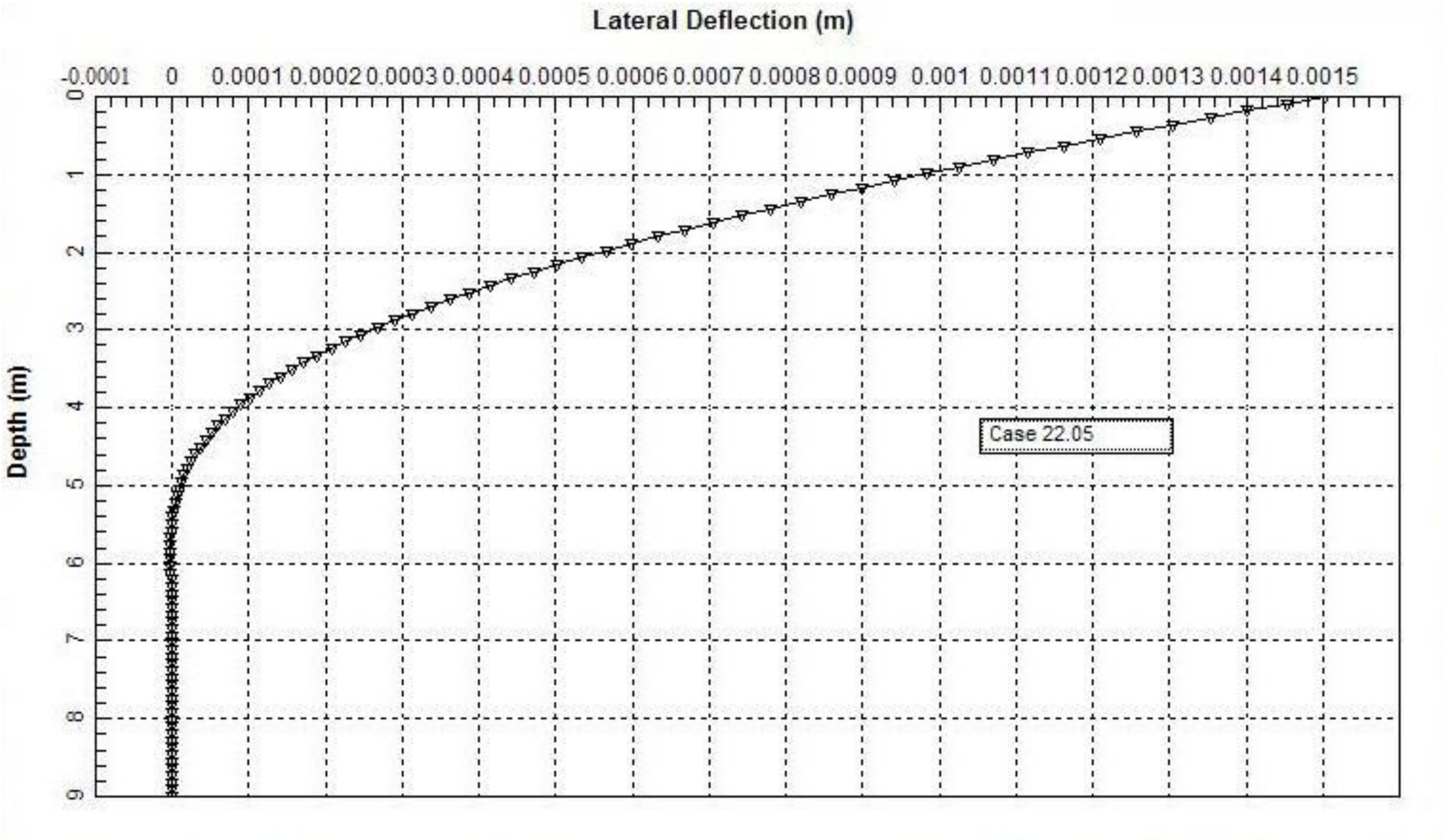
LAMPIRAN 11

OUTPUT PROGRAM *LPILE Plus* 4.0

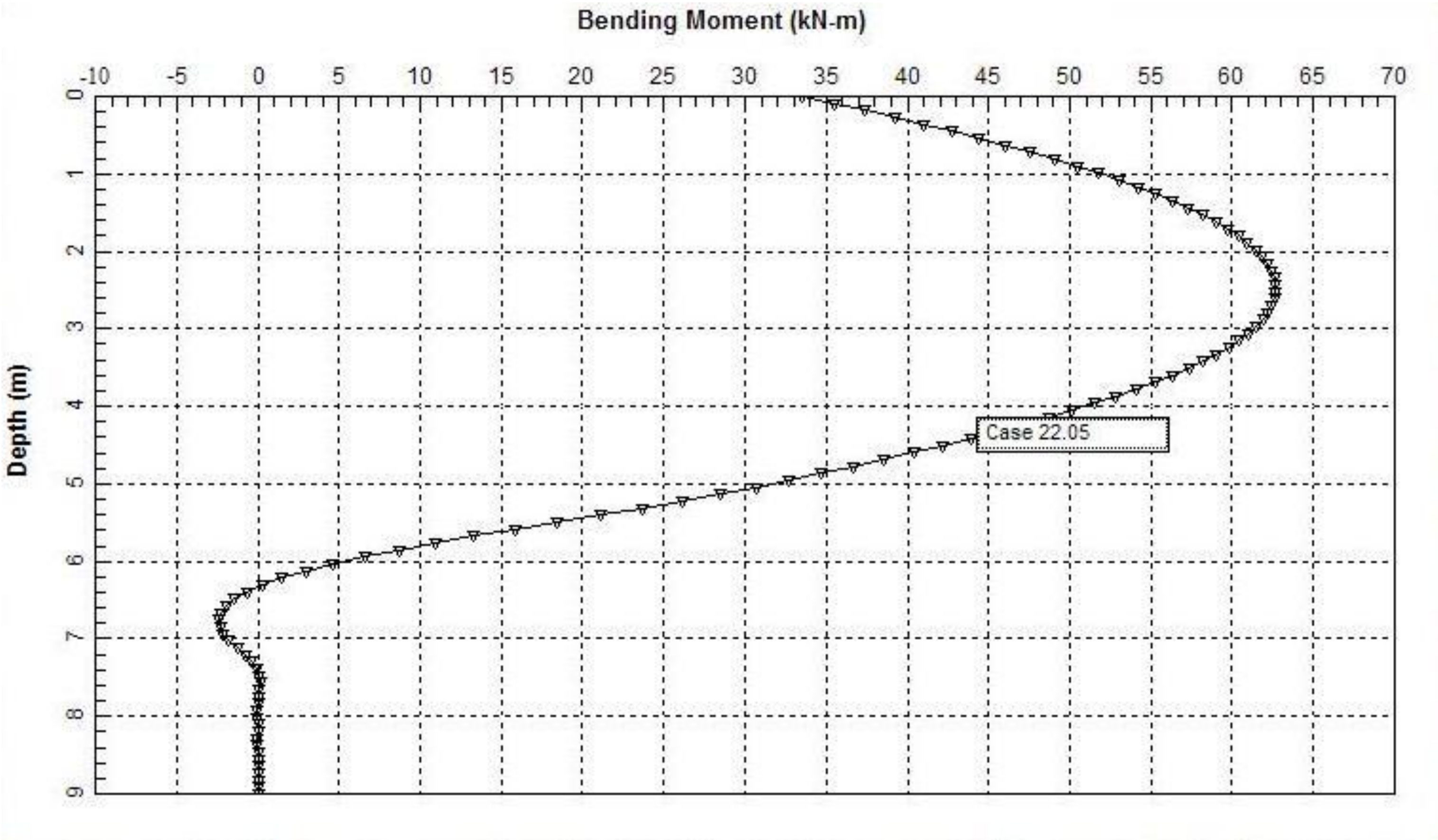
1. Metode A



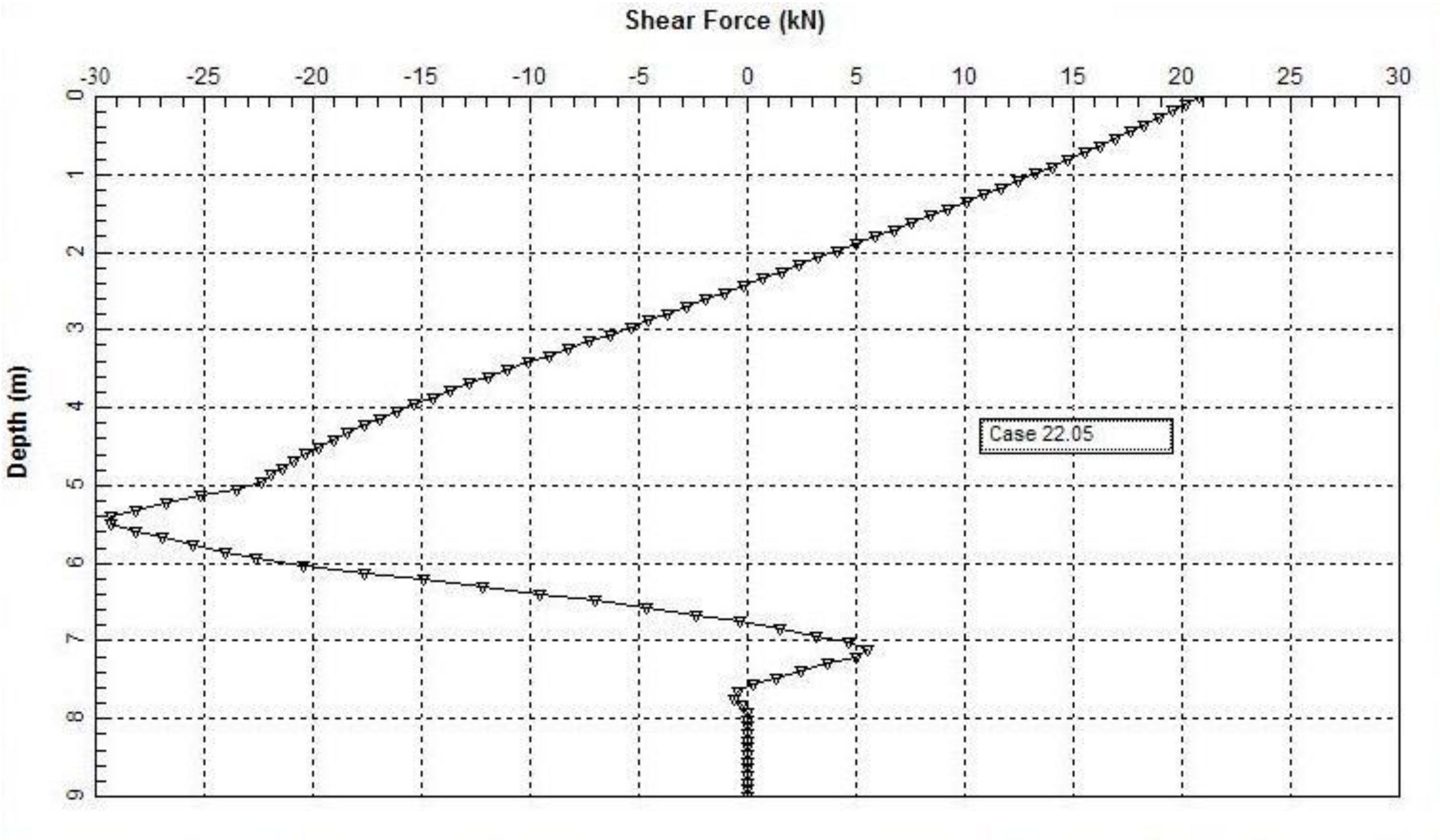
Gambar L11.1 Grafik Hubungan p-y Metode A



Gambar L11.2 Grafik *Lateral Deflection* Metode A

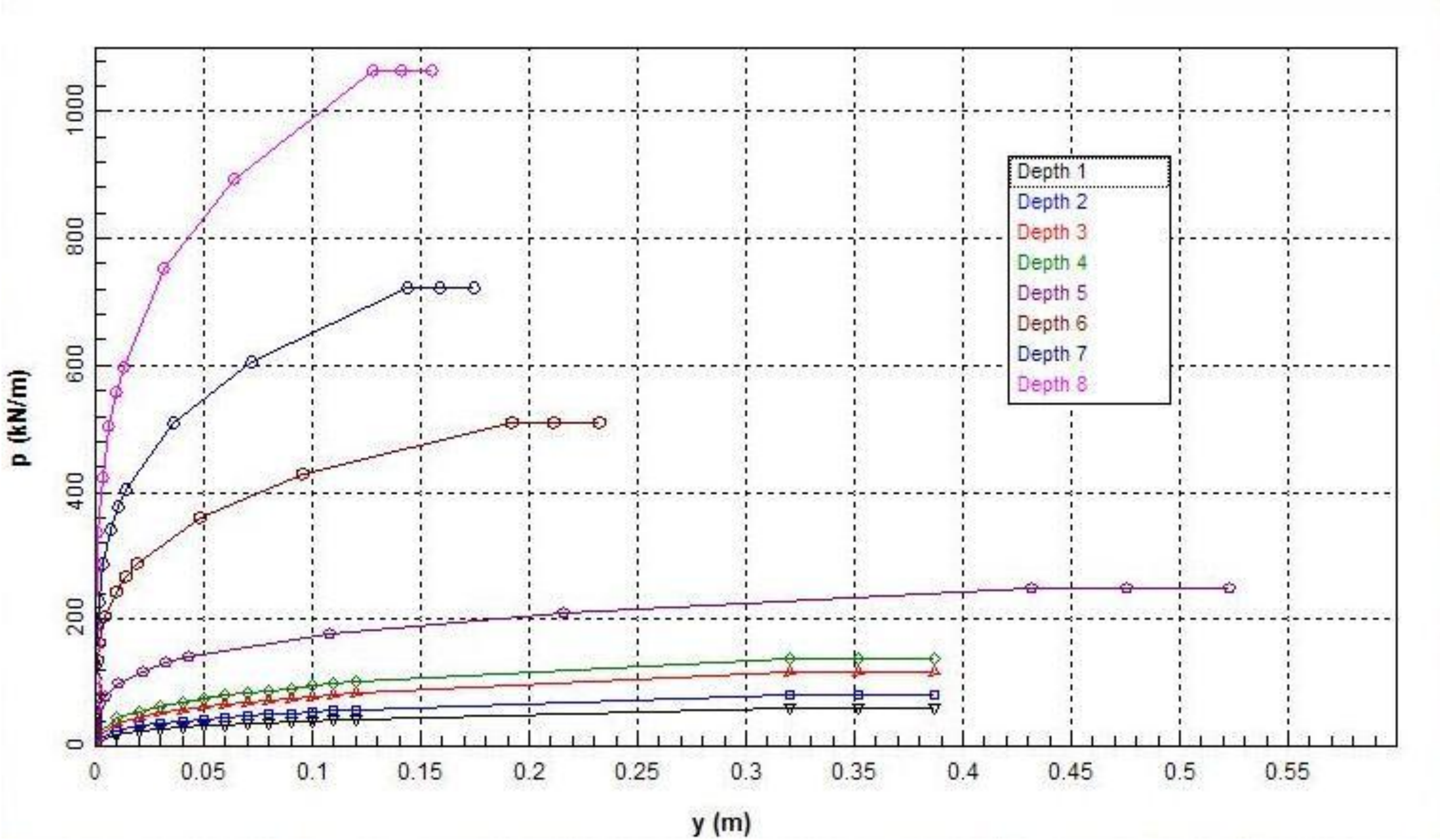


Gambar L11.3 Grafik *Bending Moment* Metode A

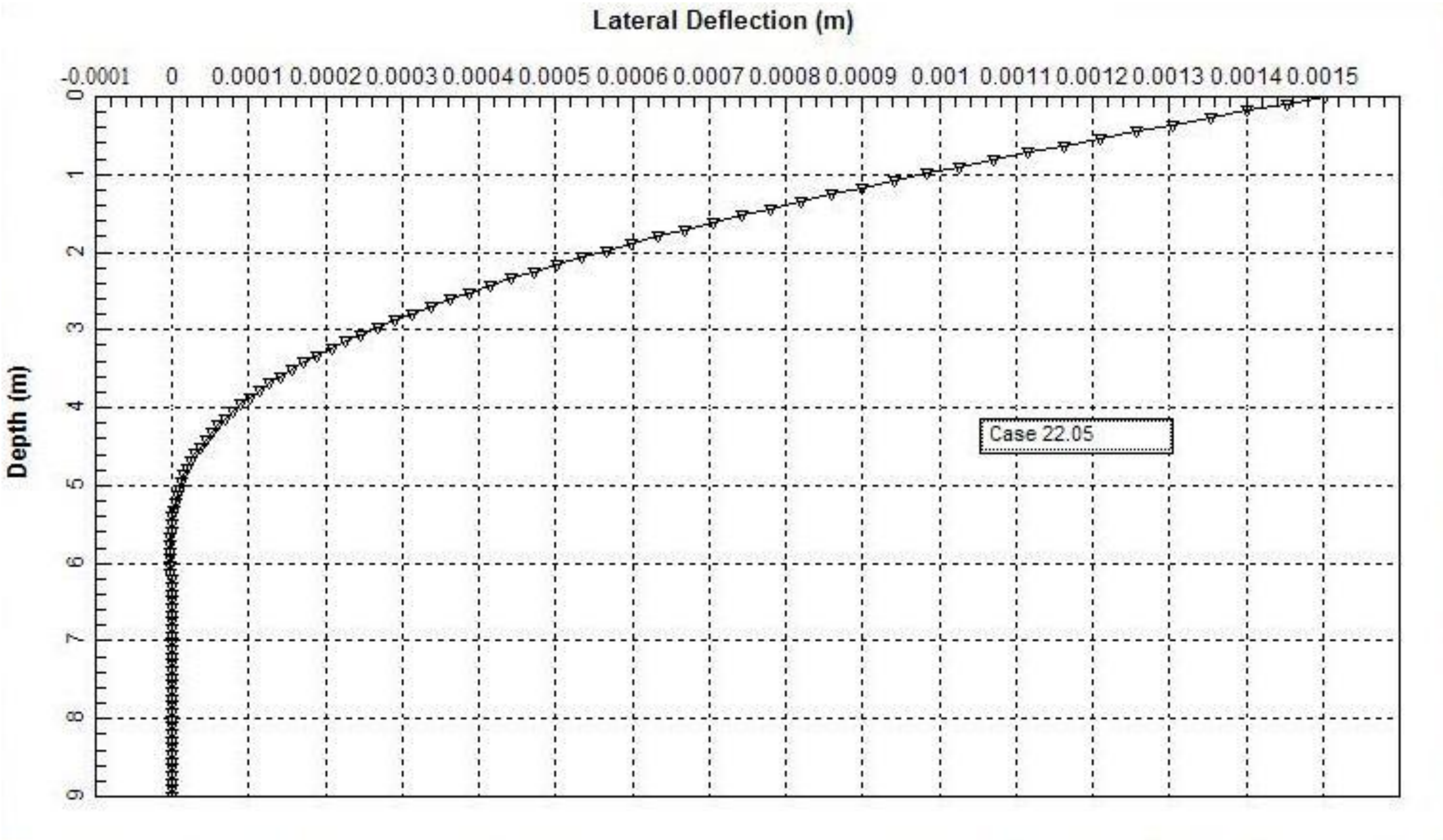


Gambar L11.4 Grafik *Shear Force* Metode A

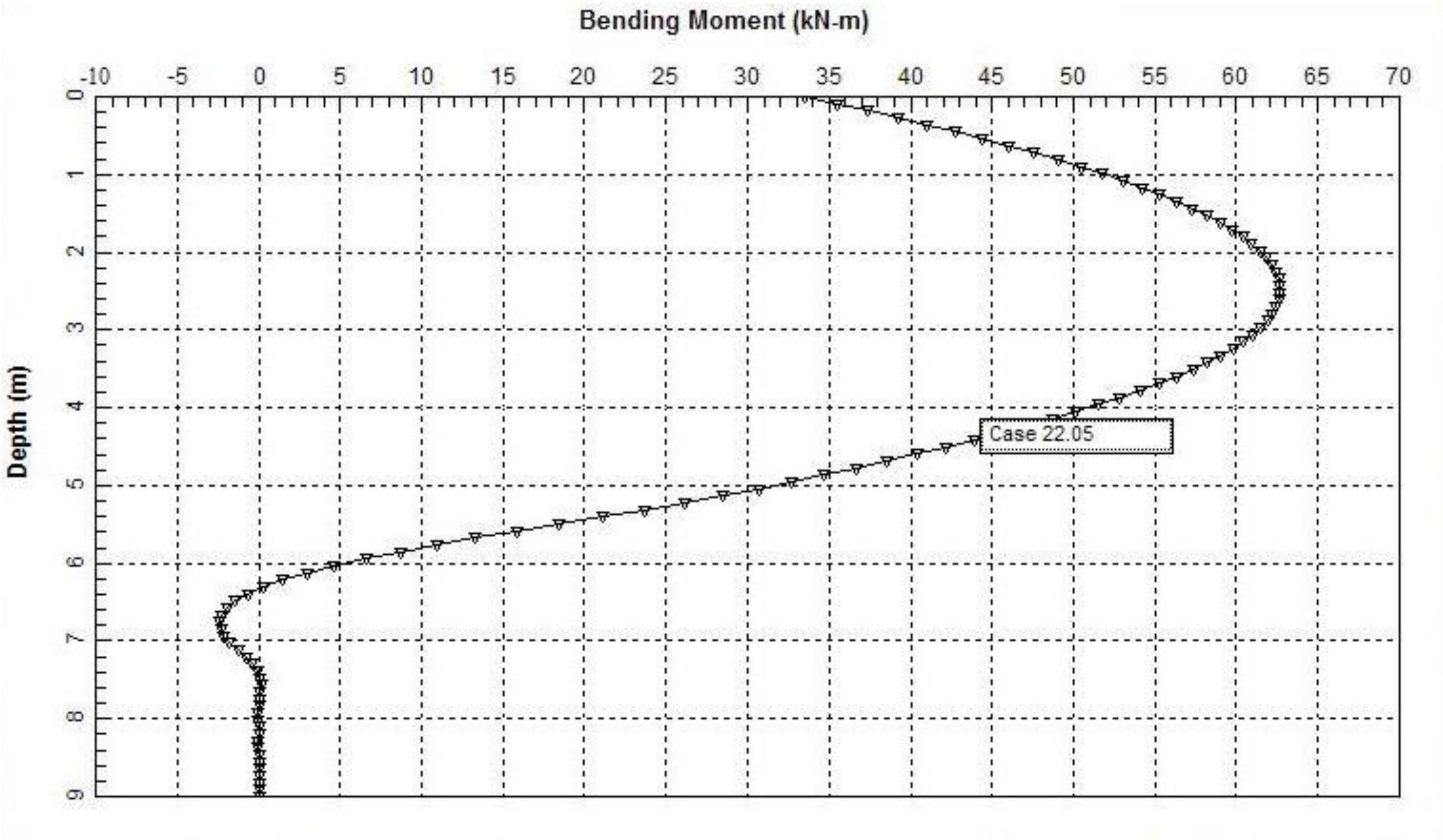
2. Metode B



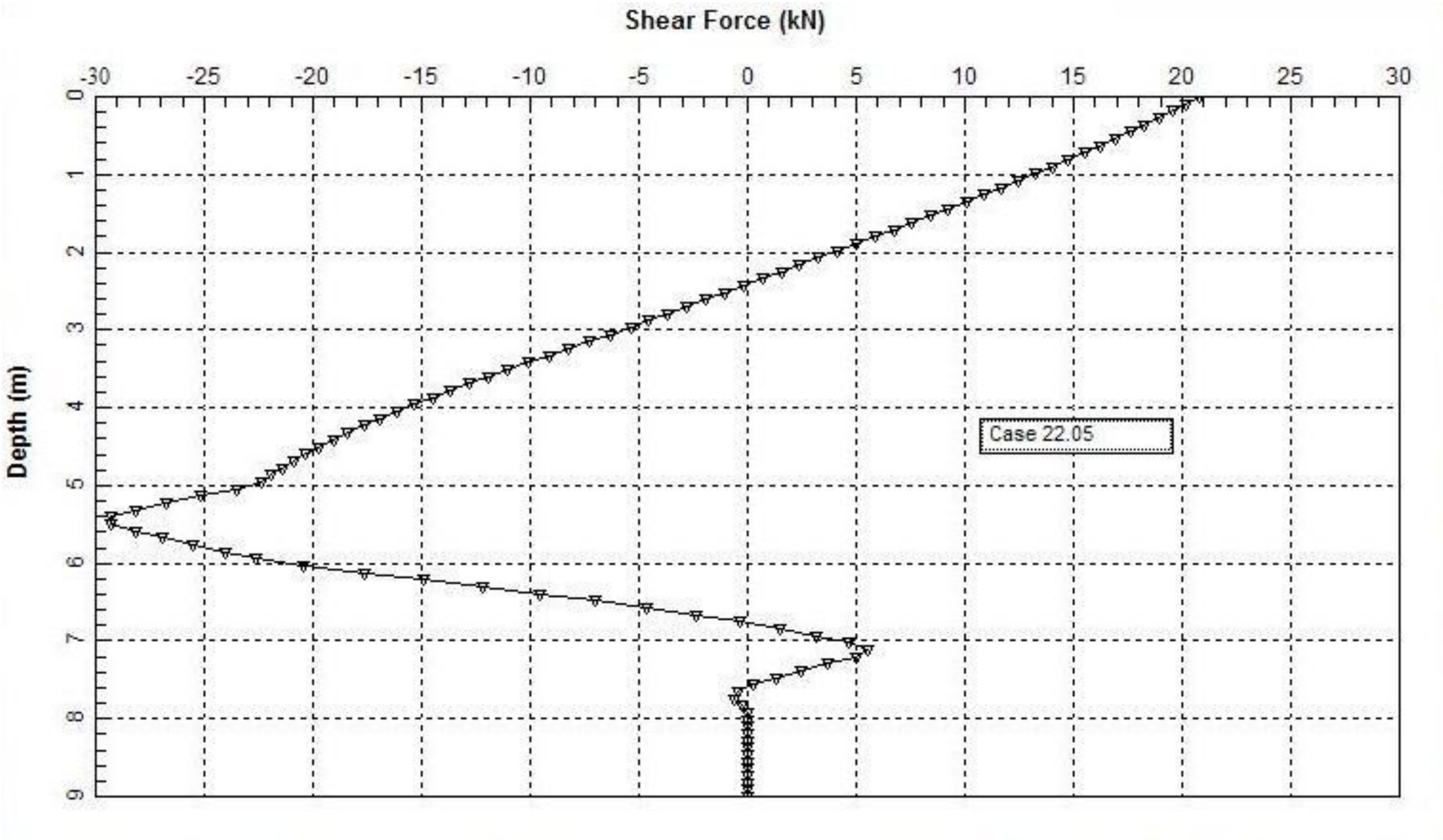
Gambar L11.5 Grafik Hubungan p-y Metode B



Gambar L11.6 Grafik *Lateral Deflection* Metode B

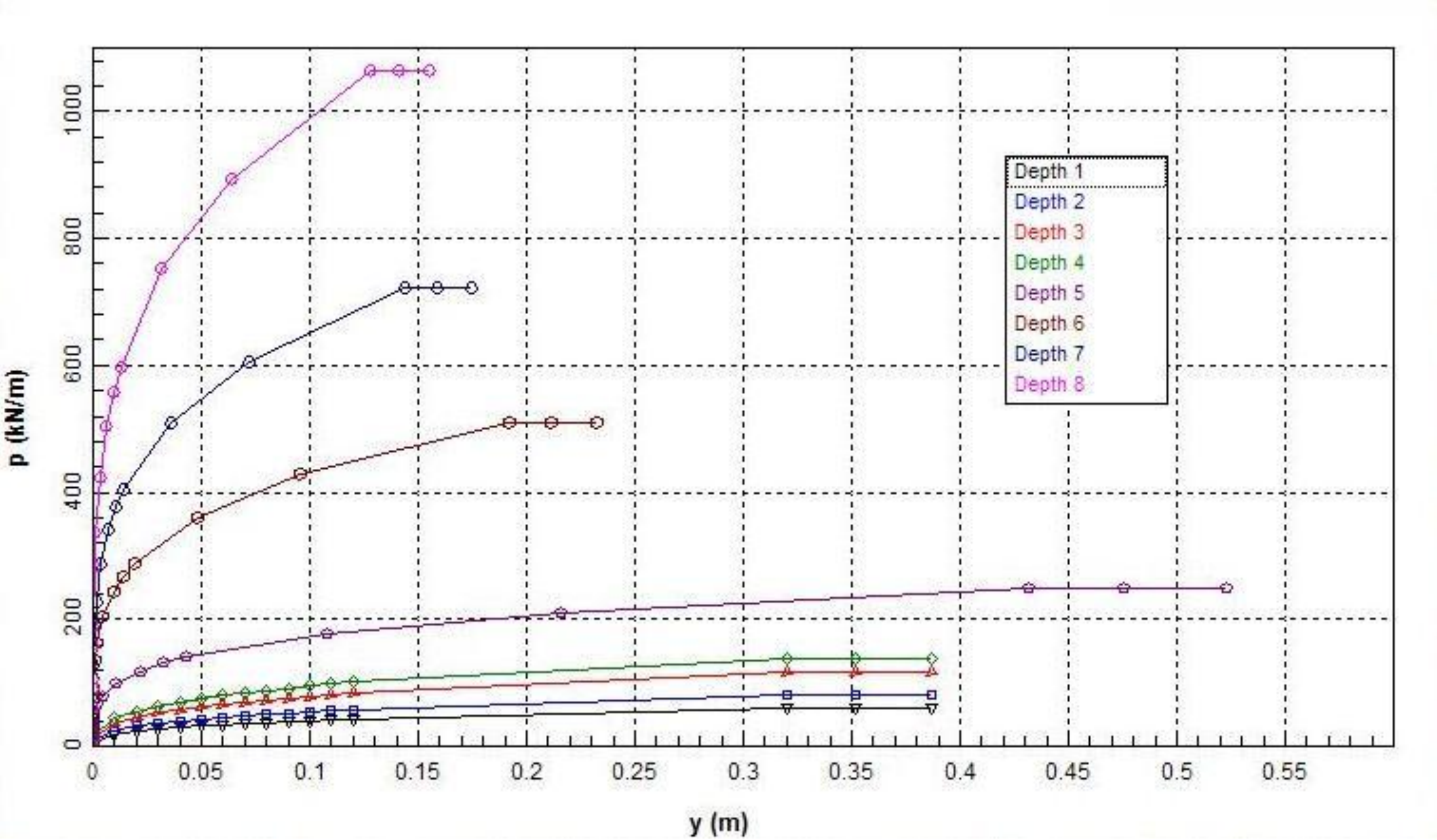


Gambar L11.7 Grafik *Bending Moment* Metode B

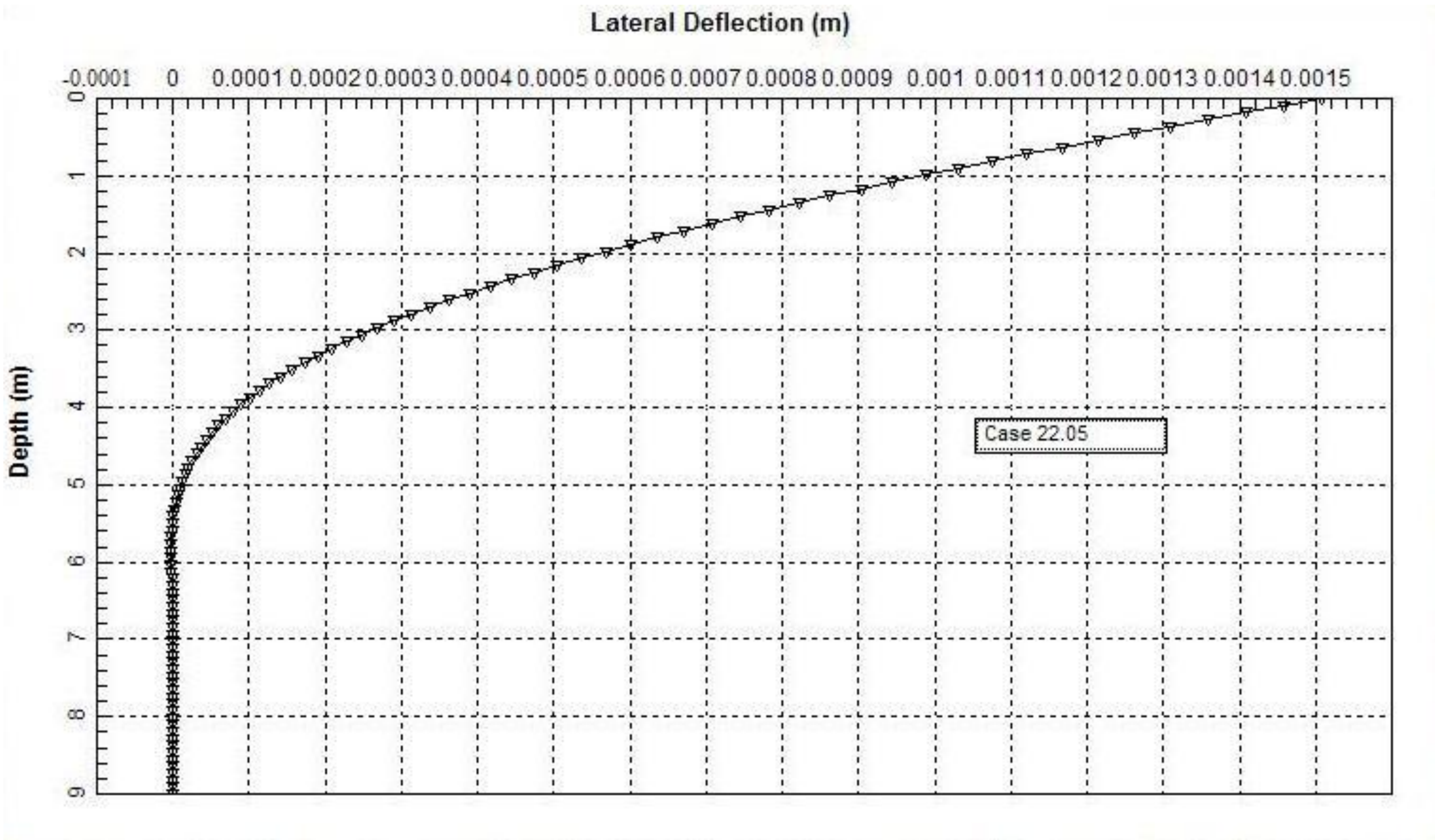


Gambar L11.8 Grafik *Shear Force* Metode B

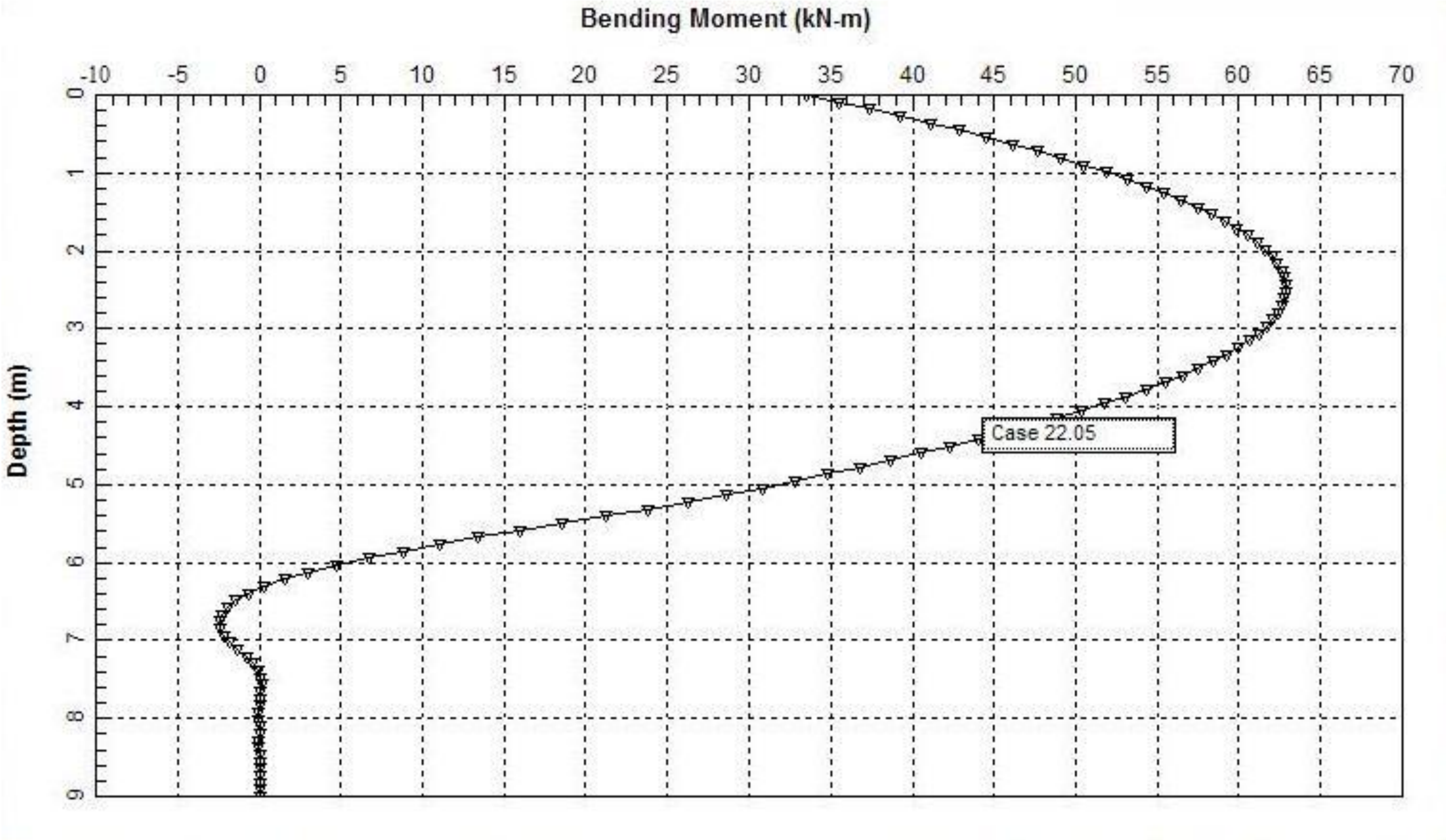
3. Metode C



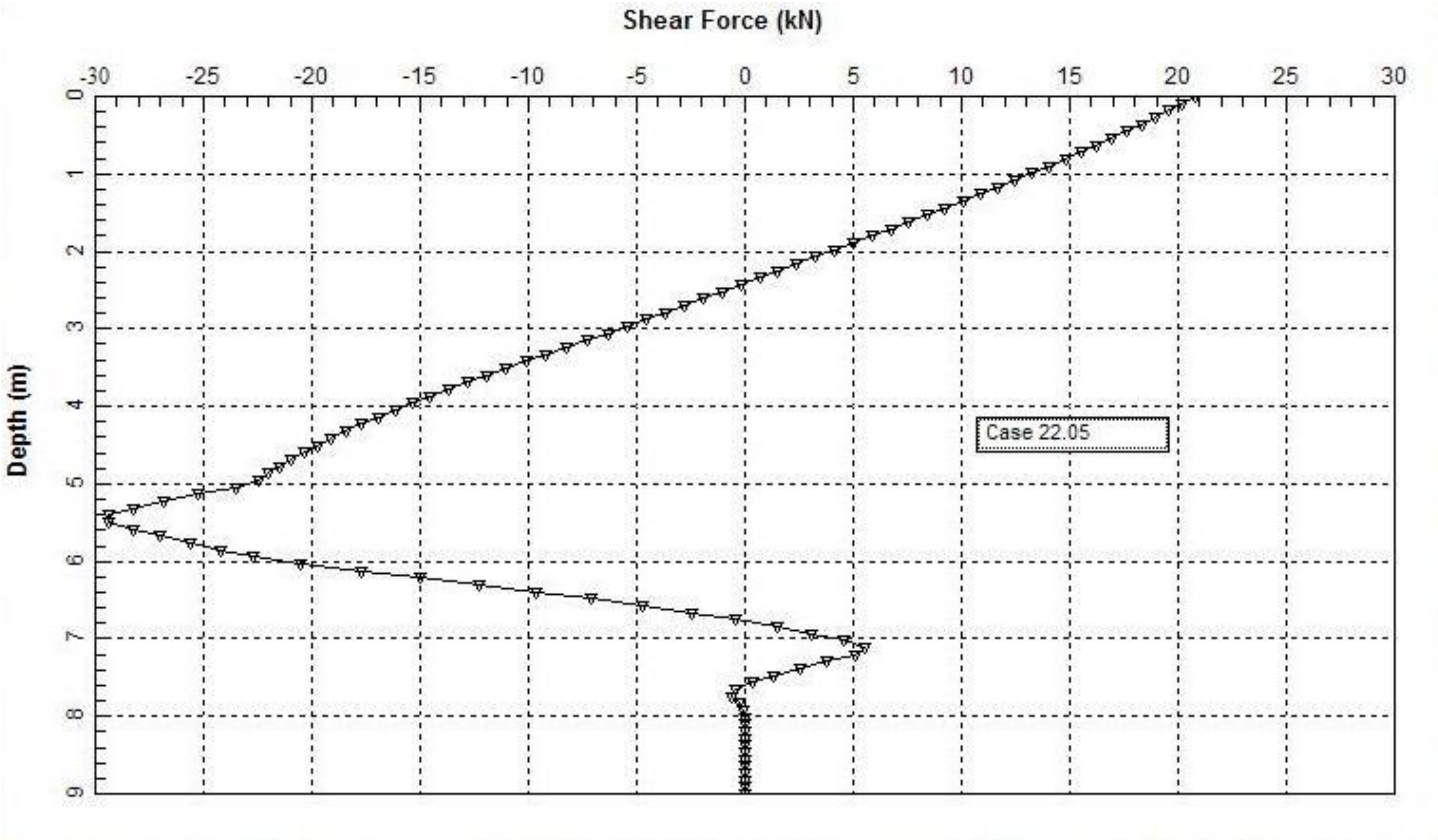
Gambar L11.9 Grafik Hubungan p-y Metode C



Gambar L11.10 Grafik *Lateral Deflection* Metode C

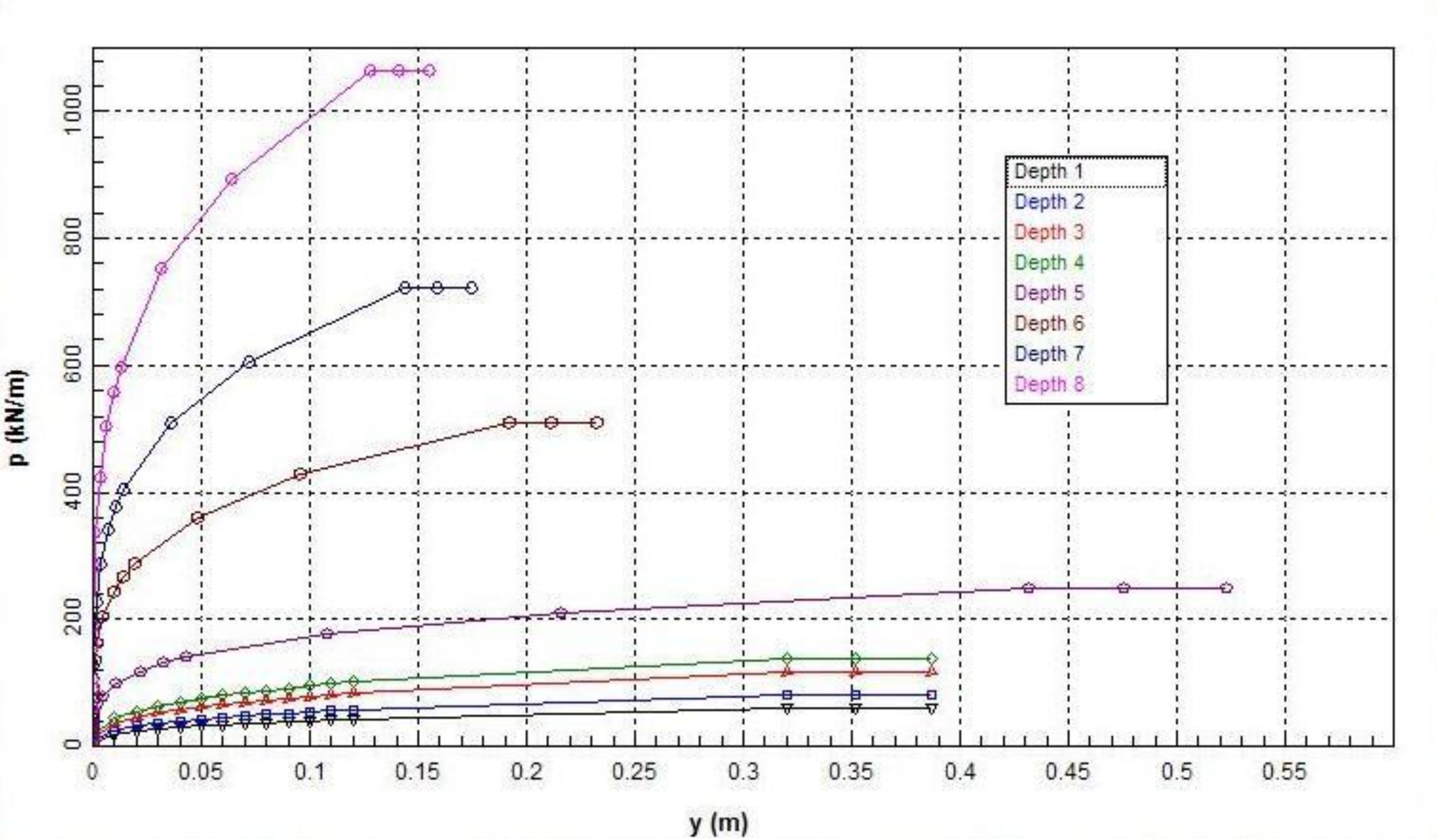


Gambar L11.11 Grafik *Bending Moment* Metode C

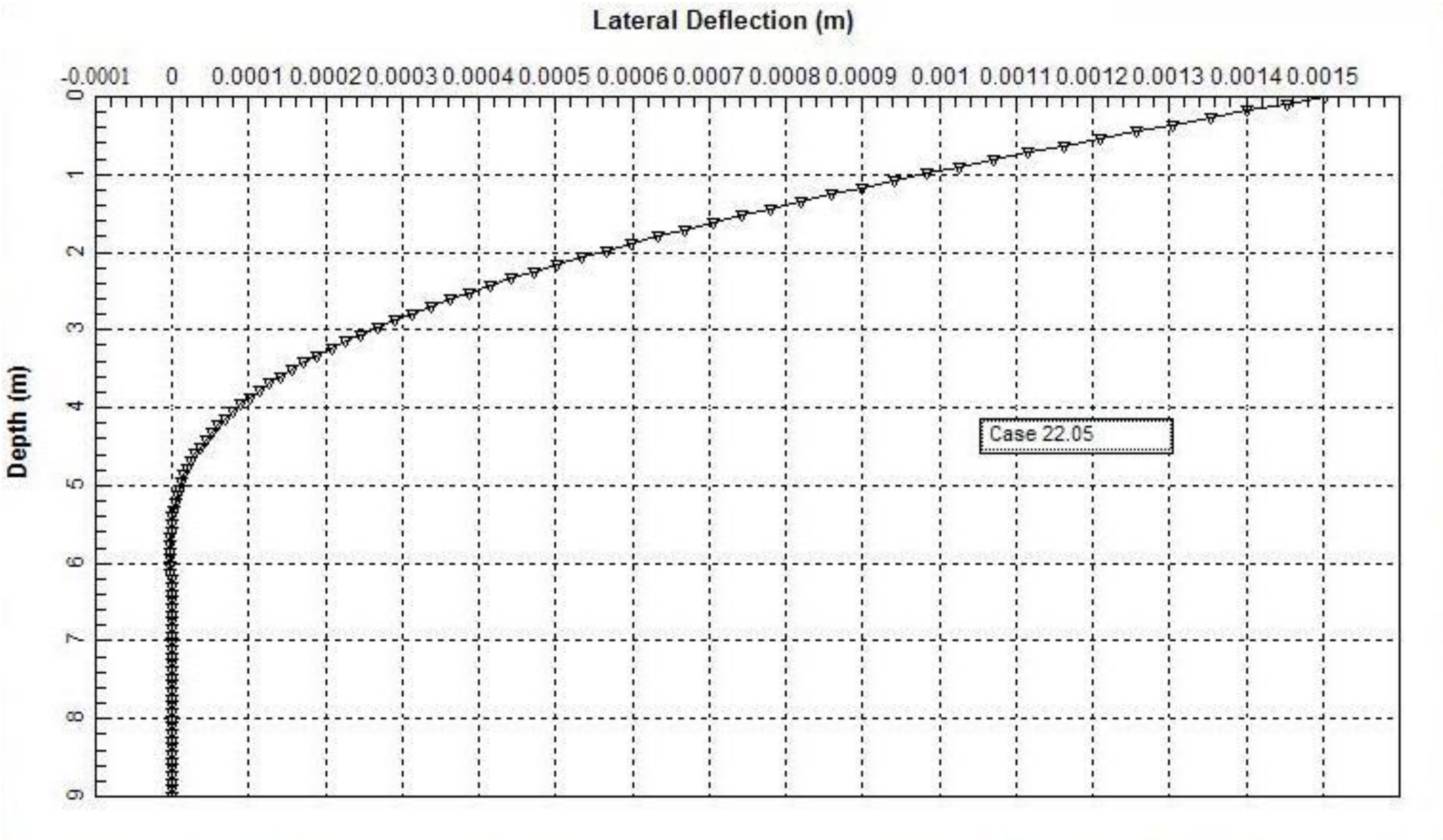


Gambar L11.12 Grafik *Shear Force* Metode C

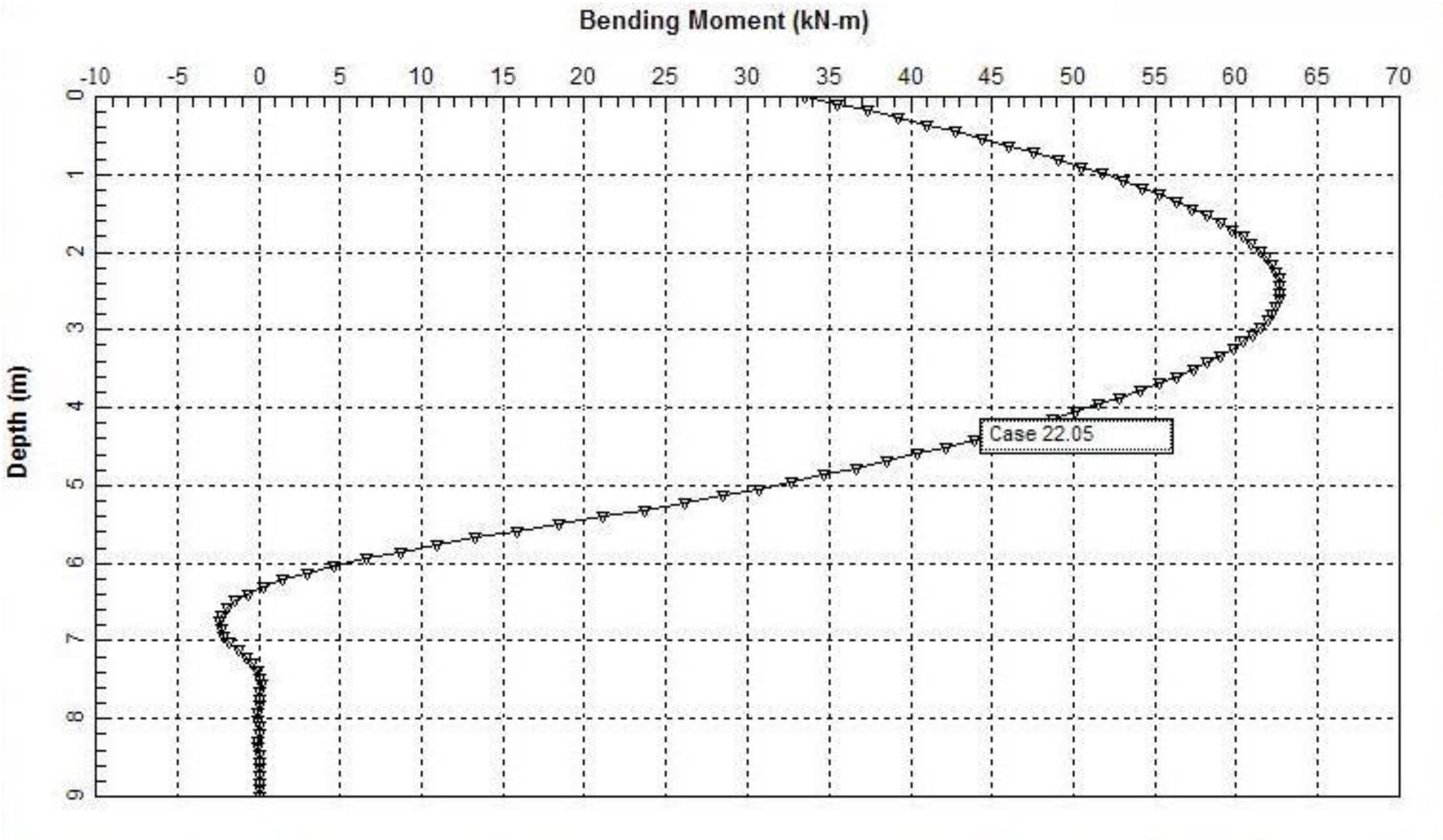
4. **Metode D**



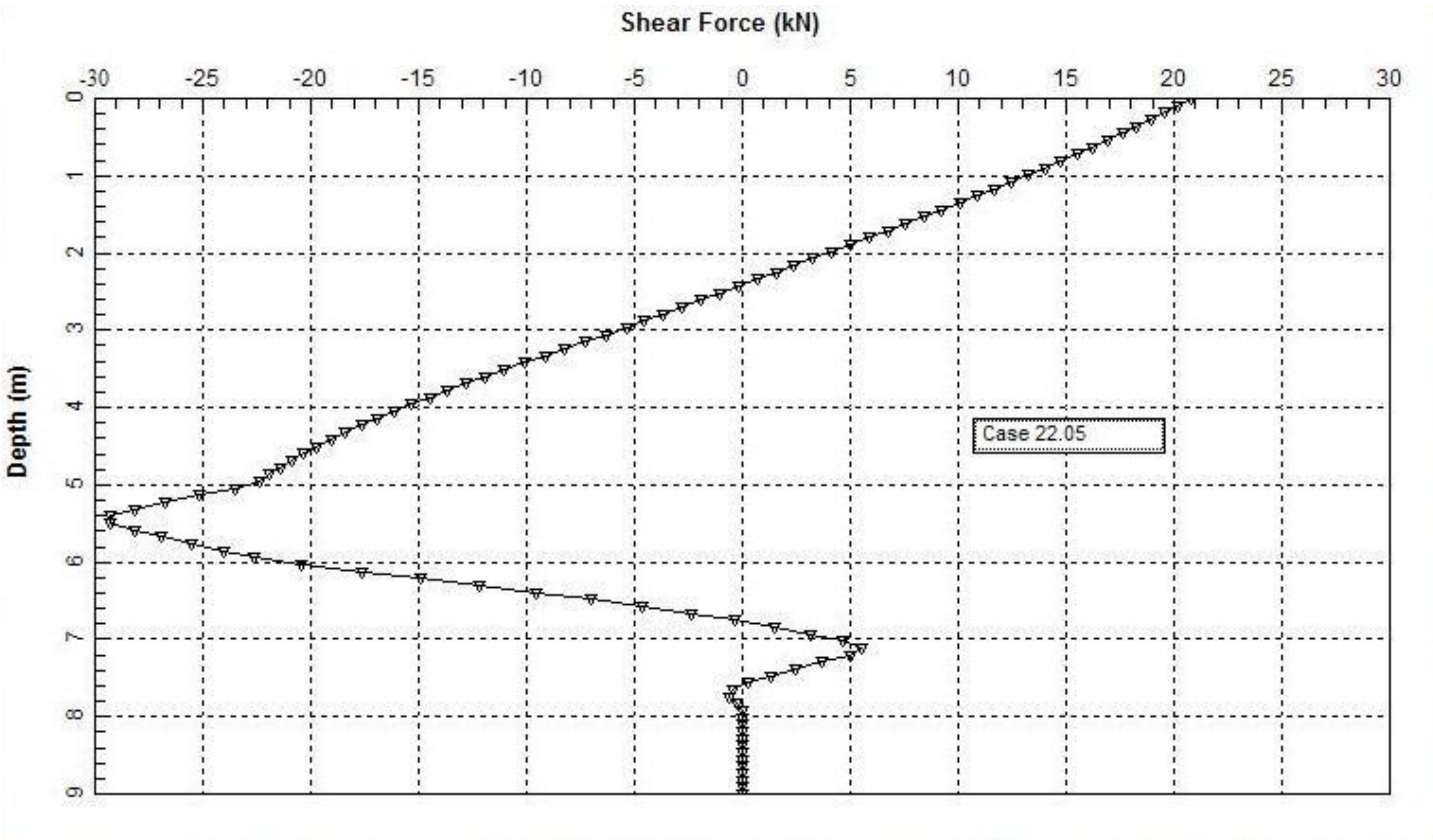
Gambar L11.13 Grafik Hubungan p-y Metode D



Gambar L11.14 Grafik *Lateral Deflection* Metode D



Gambar L11.15 Grafik *Bending Moment* Metode D



Gambar L11.16 Grafik *Shear Force* Metode D