１０．鉄塔基礎

１０．１　地質概要

地質概要の詳細は、当プロジェクトの地質調査報告書を参照されたい。

鉄塔建設ルートはウエスト・ワーフ火力発電所より起点となる鉄塔まで約 1.3kmの地中線を経由し、起点鉄塔より海岸線側を経過し、パキスタン空軍（P.A.F）基地をとり巻く様に内陸側へ通じ、バルディア変電所へ至る。

地質条件は上記の鉄塔ルートに沿って海岸線側で２つの地域（海岸線側Ⅰ，Ⅱ）と内陸側の計３つの地域に大別される。（ルート図参照）海岸線側では TL-1，TL-2，TL-5，TL-6，TL-7，TL-8，TL-9 の７地点でボーリング調査を行なっている。海岸線側Ⅰは、TL-1，TL-2，TL-5 の３地点で、細砂，砂質シルト，シルト質粘度を主体とし支持層までの深さは約10m～15mと認められる。海岸線側Ⅱは、TL-6，TL-7，TL-8，TL-9 の４地点で上層に細～粗砂、その下位にシルト質頁岩という構成となっている。支持層までの深さは浅く、深度 1.0m～2.0m程度でＮ値約 20～60となっている。なお海岸線側Ⅰ，Ⅱでは地下水位がＧＬ－0.3m～3.0m程度と高い。

内陸側は、TL-10，TL-11，TL-12 の３地点でボーリング調査を行っており、表層にレキ混り粗砂，砂質シルト、その下位にはシルト質頁岩，レキ岩，石灰岩が認められ、深度 1.0mでＮ値60程度を示し、直接基礎に適した地質構成となっている。また、地下水位は認められない。

１０．２　鉄　　塔

１０．２．１　鉄 塔 型

鉄塔型は、以下の10型に分類される。

220kV－2^{cct}	ＡＳ型，Ａ型，ＡＬ型，Ｂ型，Ｃ型，Ｄ型，ＤＲ型
220kV－2^{cct} 132kV－2^{cct}	Ａ４型，Ｄ４型，ＤＲ４型

１０．２．２　荷　　重

荷重は上部構造より伝達される。常時荷重は風速 38m/s の条件で設計されたものであり、異常時荷重は断線時の荷重である。基礎設計に使用する荷重は、常時荷重と異常時荷重を 1/1.5 した荷重とを比較し、大きい方を採用する。

荷重は、各鉄塔型について以下に示す通りである。

表 10.2.1 荷重条件

(ton)

鉄塔型	圧縮力	引揚力
ＡＳ型	37.54	25.49
Ａ型	49.27	34.09
ＡＬ型	57.95	40.17
Ｂ型	55.84	41.65
Ｃ型	85.59	68.77
Ｄ型	95.63	80.88
ＤＲ型	124.05	107.47
Ａ４型	110.95	84.56
Ｄ４型	207.39	171.08
ＤＲ４型	245.32	201.88

１０．３ 地質条件

１０．３．１ 概　要

地質条件は基礎形状を決める上で重要な要素となる。地質条件はボーリング調査により明らかにされ、地質の硬軟は標準貫入試験等の現場試験の結果より判別する。また、孔内水位は、基礎設計において考慮する。特に標準貫入試験は、杭基礎とするか、または直接基礎とするかの判断材料となり、一般に粘性土の場合でＮ値２以下、砂質土の場合でＮ値５以下程度の軟らかい地層が基礎体の床付け面より下に続く場合には、直接基礎は上部構造からの荷重に十分に耐えられず、構造物としての安定を保つことが出来ない。このような地層では、上部構造の安全性を確保する上で杭基礎が必要である。

１０．３．２ 土の物性値

設計計算に使用する土の物性値（粘着力，内部摩擦角，応力－ひずみ曲線）はＮ値より以下に示す提案式によって推定する。

$qu = N/8$　　Ｎ　：標準貫入試験によるＮ値
　　　　　　　ｑｕ：一軸圧縮強度 (kg/cm^2)

$C = qu/2$　　Ｃ　：粘着力 (kg/cm^2)

表－10.3.1　粘性土の状態とｑｕ値とＮ値の関係

	非常に軟かい	軟かい	ややかたい	かたい	非常にかたい	固　結
Ｎ　値	2以下	2～8	4～8	8～15	15～30	30以上
ｑｕ値 (kg/cm^2)	0.25以下	0.25～0.5	0.5～1.0	1.0～2.0	2.0～4.0	4.0以上

テルツァギ，ペックの提案による。

内部摩擦角とＮ値との関係を以下に示す。

$$\phi = \sqrt{15N} + 15$$

ϕ ：内部摩擦角（度）

直接基礎及び杭基礎の支持力は土の物性値より求められる。

今回設計に使用するボーリング調査番号、及び物性値は地質調査報告書より次の通りとする。

表－10.3.2　代表ボーリングＮＯ及び物性値

基　礎　型	ボーリングNo	粘着力 C (t/m^2)	内部摩擦角 ϕ（度）	土の単位体積重量 γ (t/m^3)
杭基礎（海岸線側Ｉ）	ＴＬ－１	0	20	1.8
直接基礎（海岸線側II）	ＴＬ－６	0	35	1.8
直接基礎（内　陸　側）	ＴＬ－12	0	40	1.8

尚、直接基礎（内陸側）の床付位置の岩盤層においては、地質調査報告書の一軸圧縮試験結果（ｑｕ値）より、τ（せん断強度）が十分期待できることから、引揚支持力算定の際 $\tau = 5t/m^2$ を床板側面に考慮することとした。

１０．３．３　地下水位

地下水位は基礎形状を決める上で、重要な要素となる。

特に海岸線側Ｉ，IIでは、地下水位が高いため、地下水位を考慮した設計を行なわなければならない。地下水位による浮力は引揚力に抵抗する基礎体コンクリートや床板上の土の重量を軽減させ、引揚支持力の低下につながる。よって海岸線側Ｉ，IIの基礎形状は地下水位を考慮しない内陸側に比べ大きくなり、経済的に不利となるだけでなく、床付面が深くなり、施工的にも困難になる。

今回の設計に使用する地下水位は地質調査報告書より次の通りとする。

表－10.3.3　設 計 地 下 水 位

基礎型	地　域	設計地下水位（m）
杭基礎	海岸線側Ⅰ	ＧＬ±０
直接基礎	海岸線側Ⅱ	ＧＬ±０
直接基礎	内 陸 側	地下水位なし

１０．４ 基礎型

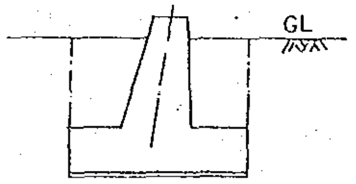
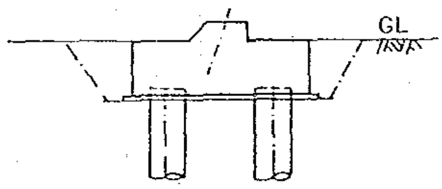
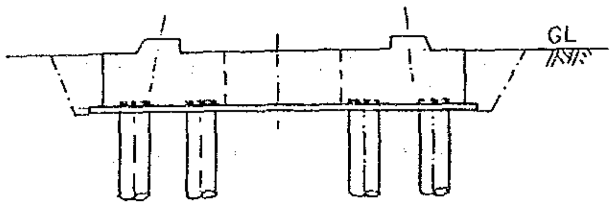
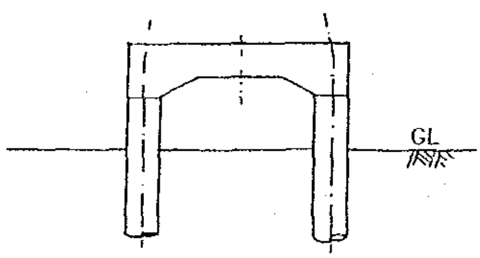
鉄塔基礎は２つのタイプに大別される。１つは直接基礎であり、鉄塔からの荷重を直接支持地盤に伝達する。もう１つは杭基礎であり、一般に支持層が深い場合に用いられる。基礎型を以下に示す。

- 直接基礎
  - Ⅰ型（独立型）
  - Ⅳ-1型（マットタイプ）
- 杭基礎
  - Ⅲ型（独立型）
  - Ⅳ-2型（一体型）
  - ラーメン型

今回の設計では、基礎型比較検討の結果、直接基礎ではⅠ型，杭基礎ではⅢ型を採用した。

又、海岸線側Ⅰ区間，LAYARI RIVER 横断部は、洪水水位を考慮し、ラーメン型基礎とする。

表−10.4.1　基 礎 型 形 式 図

基礎形式 ＼ 項		形　状　図
直接基礎	Ⅰ　型	
	Ⅳ −1 型	
杭基礎	Ⅲ　型	
	Ⅳ −2 型	
	ラーメン型	

１０．５ 杭

杭は、軟弱地盤で支持層の深い場所、又は直接基礎では安定性が保てない場合に用いられる。杭は以下の様に分類される。

〔材料による分類〕

打込み杭
- コンクリート杭
- ＰＣ杭
- 鋼管杭

場所打杭
- 場所打コンクリート杭

〔機能による分類〕

杭
- 支持杭
- 摩擦杭

支持杭は上部構造の荷重を軟らかい層をつらぬき、支持層となる岩盤や、堅い地盤に伝えるものである。支持層は十分な強度が期待でき、一般に粘性土でＮ値10以上、砂質度でＮ値30以上の層を支持層とする。また摩擦杭は比較的荷重の小さい場合に用いられ、地盤と杭周面の摩擦によって荷重を地盤に伝えるものである。

杭種の選択は土質条件，荷重条件，杭体の強度，経済性等によって決められる。

今回の設計では、現地での施工実績から杭径 22ｲﾝﾁ（ϕ559mm）の場所打ちコンクリート杭を採用することとした。なお、ラーメン型基礎及び荷重の大きいＤ４型，ＤＲ４型は杭径 36ｲﾝﾁ（ϕ914mm）の場所打ちコンクリート杭を採用した。

又、地質調査の結果、支持層が10m～15mと杭基礎としては比較的浅いため、全て支持杭として設計を行なった。

１０．６　鉄塔基礎型比較検討

比較検討は、荷重条件，地質条件によりグループ分けし、各グループについて基礎型の比較検討を行った。その結果経済性から各グループの最適な基礎型は、下記の通とする。

１０．６．１　比較検討結果

表－10.6.1　比較検討結果一覧

地　域	鉄塔型グループ	検討代表鉄塔型	比較基礎型	最適基礎型	備　考
内陸側 海岸線側 Ⅱ	As，A	As	Ⅰ型，Ⅳ$_{-1}$型	Ⅰ 型	地下水位なし（内陸側）
					地下水位GL±０（海岸線側Ⅱ）
	D，B C，DR	D	Ⅰ型，Ⅳ$_{-1}$型	Ⅰ 型	地下水位なし（内陸側）
					地下水位GL±０（海岸線側Ⅱ）
海岸線側 Ⅰ	A4，AL	A4	Ⅲ型，Ⅳ$_{-2}$	Ⅲ 型	地下水位GL±０（海岸線側Ⅰ）
	DR4，D4	DR4	Ⅲ型，Ⅳ$_{-2}$	Ⅲ 型	地下水位GL±０（海岸線側Ⅰ）

尚、鉄塔型ＡＬ型で海岸線側Ⅰ区間 LAYARI RIVER 横断部は、洪水水位を考慮し、ラーメン型基礎とし、比較検討より除く。

１０．６．２　各鉄塔型による基礎型コメント

ＡＳ型 …… 鉄塔建設地点は海岸線側Ⅱと内陸側に分かれる。比較検討の結果、Ⅰ型を採用する。なお、内陸側については海岸線側Ⅱと同傾向にあるため比較検討を省略した。

表－10.6.2　　ＡＳ型におけるⅠ型とⅣ$_{-1}$型の経済比較（海岸線側Ⅱ）

種　別	工　　種	単　　価 (RS／m^3)	数　　量 (m^3)	概算工事費 (ＲＳ)
Ⅰ　型	掘　　削	100	234	168,800
	コンクリート	2000	72.7	
Ⅴ$_{-1}$型	掘　　削	50	159	346,100
	コンクリート	3000	112.7	

Ａ　型 …… 鉄塔建設地点は主に内陸側である。ＡＳ型の比較検討結果を踏まえ、Ⅰ型を採用する。

ＡＬ型 …… 鉄塔建設地点の地質条件より杭基礎とし、Ａ４型の比較検討結果を踏まえ、Ⅲ型を採用する。また、ラーメン型基礎を含む。

Ｂ　型 …… 鉄塔建設地点は海岸線側Ⅱと内陸側に別れる。D型の比較検討結果を踏まえ、Ⅰ型を採用する。

Ｃ　型 …… 鉄塔建設地点は海岸線側Ⅱと内陸側に分かれる。D型の比較検討結果を踏まえ、Ⅰ型を採用する。

Ｄ　型 …… 鉄塔建設地点は海岸線側Ⅱと内陸側に分かれる。比較検討の結果Ⅰ型を採用する。

表－10.6.3　D型におけるⅠ型とⅣ-1型の経済比較（海岸線側Ⅱ）

種別	工種	単価 (RS／㎥)	数量 (㎥)	概算工事費 (RS)
Ⅰ型	掘削	100	633	440,300
	コンクリート	2000	188.5	
Ⅳ-1型	掘削	50	400	929,900
	コンクリート	3000	303.3	

DR型 ……鉄塔建設地点は海岸線側Ⅱと内陸側に分かれる。D型の比較検討結果を踏まえ、Ⅰ型を採用する。

A4型 ……鉄塔建設地点は海岸線側Ⅰである。地質条件より杭基礎とする。比較検討の結果、Ⅲ型を採用する。

表－10.6.4　A4型におけるⅢ型とⅣ-2型の経済比較（海岸線側Ⅰ）

種別	工種	単価 (RS／㎥)	数量 (㎥)	概算工事費 (RS)
Ⅰ型	掘削	50	342	1,023,900
	コンクリート	3000	162.8	
	杭（φ22ｲﾝﾁ）	1800	288.0	
Ⅳ-2型	掘削	50	384	1,504,800
	コンクリート	3000	264.8	
	杭（φ22ｲﾝﾁ）	1800	384.0	

D4型 ……鉄塔建設地点は海岸線側Ⅰである。地質条件より杭基礎とし、DR4型の比較検討結果を踏まえ、Ⅲ型を採用する。

DR4型 …鉄塔建設地点は海岸線側Ⅰである。地質条件より杭基礎とする比較検討の結果、Ⅲ型を採用する。

表－10.6.5　ＤＲ４型におけるⅢ型とⅣ-2型の経済比較（海岸線側Ｉ）

種　別	工　　種	単　　価 (RS／㎥)	数　　量 (㎥)	概算工事費 (ＲＳ)
Ⅲ　型	掘　　削	50	688	2,439,900
	コンクリート	3000	361.3	
	杭（φ36インチ）	5900	224.0	
Ⅳ-2型	掘　　削	50	762	2,934,000
	コンクリート	3000	533.3	
	杭（φ22インチ）	1800	720.0	

表－10.6.6　鉄 塔 基 礎 型 一 覧 表

鉄塔型	ボーリングNo.	基礎型	直接基礎	杭基礎	備　　考
ＡＳ－Ｄ	ＴＬ－12	Ｉ	○		
ＡＳ－Ｃ	ＴＬ－６	Ｉ	○		
Ａ　－Ｄ	ＴＬ－12	Ｉ	○		
ＡＬ－Ｓ	ＴＬ－１	Ⅲ		○	
ＡＬ－Ｒ	ＴＬ－１	ラーメン型		○	
Ｂ　－Ｄ	ＴＬ－12	Ｉ	○		
Ｂ　－Ｃ	ＴＬ－６	Ｉ	○		
Ｃ　－Ｄ	ＴＬ－12	Ｉ	○		
Ｃ　－Ｃ	ＴＬ－６	Ｉ	○		
Ｄ　－Ｄ	ＴＬ－12	Ｉ	○		
Ｄ　－Ｃ	ＴＬ－６	Ｉ	○		
ＤＲ－Ｄ	ＴＬ－12	Ｉ	○		
ＤＲ－Ｃ	ＴＬ－６	Ｉ	○		
Ａ４－Ｓ	ＴＬ－１	Ⅲ		○	
Ｄ４－Ｓ	ＴＬ－１	Ⅲ		○	
ＤＲ４－Ｓ	ＴＬ－１	Ⅲ		○	

注）鉄塔型のサブ番号は下記の通り

１．Ｄ　は内陸側（地下水位なし）

２．Ｓ　は海岸線側Ｉ（地下水位あり）

３．Ｃ　は海岸線側Ⅱ（地下水位あり）

４．Ｒ　はラーメン型を示す

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10.7 詳細設計

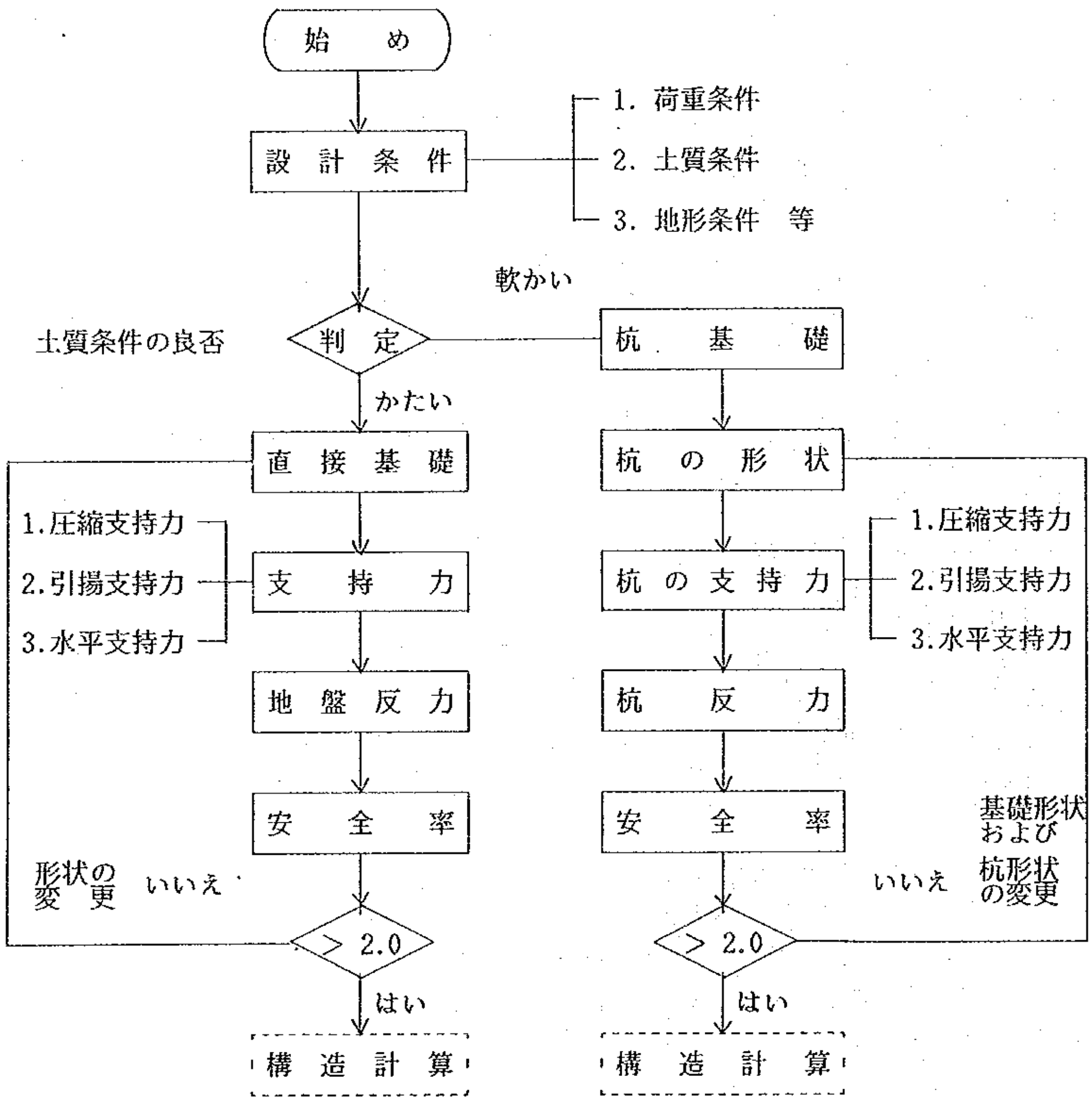
詳細設計は次頁に示すフローチャートに従い、安定計算，構造計算を行なう。

なお、ラーメン型基礎の構造計算については、フレーム解析を行い、構造を決定する。

10.7.1 設計フロー図

鉄塔基礎設計のためのフローチャート

(a) 安定計算



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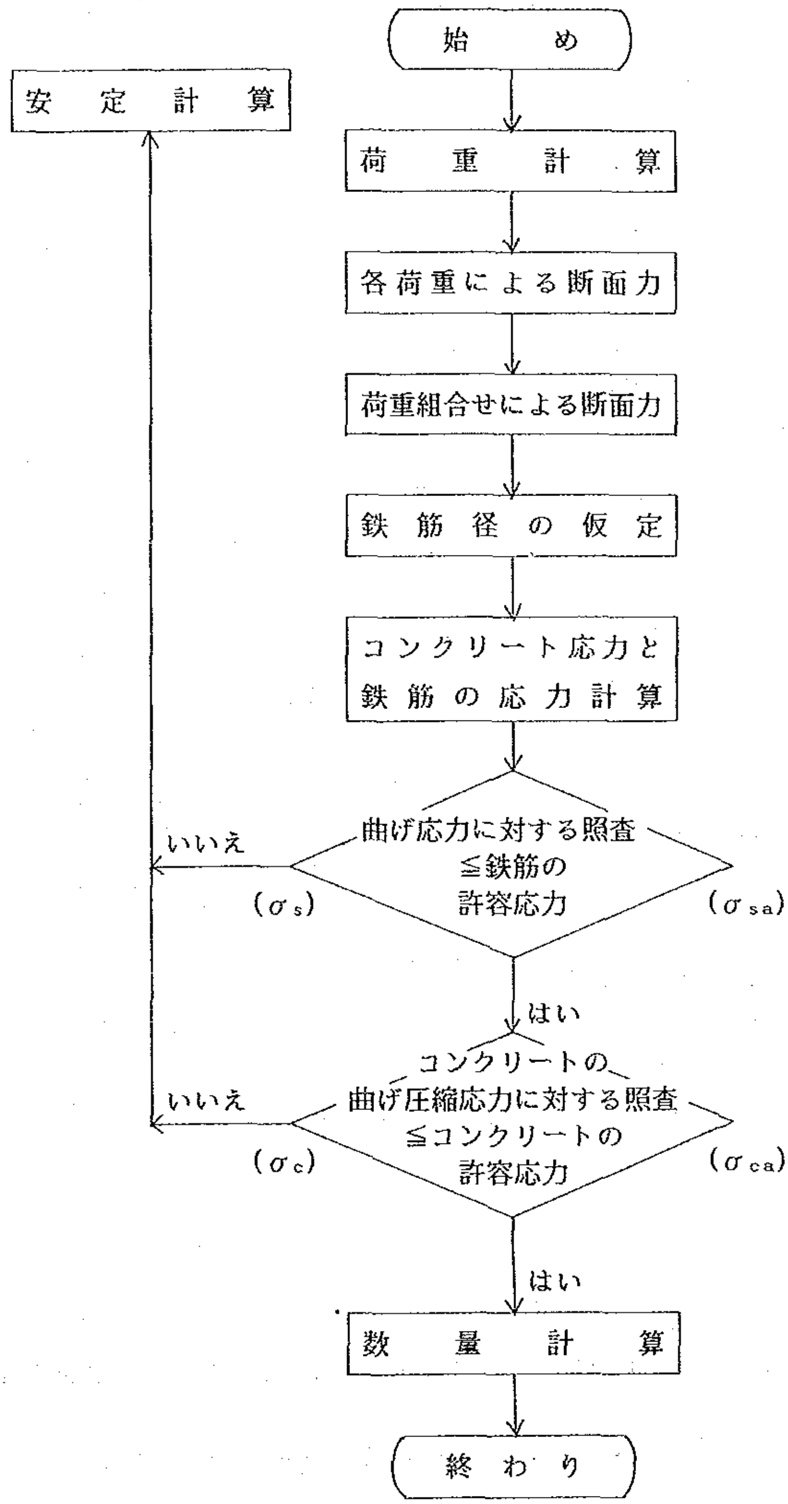
(b) 構造計算



始め
荷重計算
各荷重による断面力
荷重組合せによる断面力
鉄筋径の仮定
コンクリート応力と
鉄筋の応力計算
曲げ応力に対する照査
≦鉄筋の
許容応力
(σs)
(σsa)
いいえ
はい
コンクリートの
曲げ圧縮応力に対する照査
≦コンクリートの
許容応力
(σc)
(σca)
いいえ
はい
数量計算
終わり
安定計算

10.7.2 設計計算

(1) 基礎体寸法の制限値

基礎型の選定後、基礎体寸法を決めるに当たり、過去の実績及び研究成果より、以下に示す制限値を考慮する。(図 10.7.1)

(a) 柱　体

○天 端 巾：a

$a \geqq 3 \cdot L$ (又は $3 \cdot \phi$)

ここに

L ＝ アングル鉄塔のフランジ巾

ϕ ＝ 鋼管鉄塔の主柱材の直径

○底 面 巾：b

浅型基礎：$b \geqq a + 0.15H$

深型基礎：$b \geqq a + 0.15H$ かつ $b \geqq B/4$

ここに

H ＝ 柱体高さ

B ＝ 床 板 巾

○柱体地上高さ：f

一般に　f ＝ 30cm

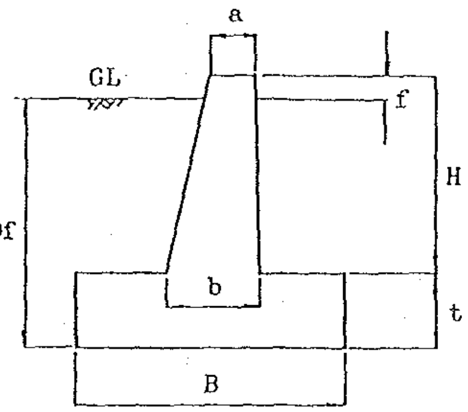


図 10.7.1

(b) 床　板

○形　状：主に正方形

○床板厚：t

$t \geqq B/5$

ただし、圧縮力が 200_{ton}以上の場合、最小厚さは1mとする。

(2) 直接基礎の安定計算

(a) 降伏圧縮支持力：qcy

基礎の支持力は、以下に示すテルツァーギィの公式によって計算する。

$$qcy = \frac{1}{1.5}\{\alpha \cdot C \cdot Nc + \beta \cdot rs_1 \cdot B \cdot Nr + rs_2 \cdot D_f \cdot Nq\}$$

ここに

　q_{cy} ＝ 降伏圧縮支持力（ton/m²）

　α, β ＝ 形状係数－表 10.7.1 に示す。

表 10.7.1　形 状 係 数

基礎の形状	連　続	正方形	長方形	円
α	1.0	1.3	$1+0.3\frac{B}{L}$	1.3
β	0.5	0.4	$0.5+0.1\frac{B}{L}$	0.3

B，L ＝ それぞれ長方形の短辺と長辺を表す。（B＜L）

C ＝ 基礎体底面下の土の粘着力（ton/m²）

r_{s1} ＝ 基礎体底面下の平均単位体積重量（ton/m³）

（ただし、地下水位下の部分は、水中重量を考慮する。）

r_{s2} ＝ 基礎体底面上の平均単位体積重量（ton/m³）

（ただし、地下水位下の部分は、水中重量を考慮する。）

D_f ＝ 地表より基礎体底面までの根入深さ（m）

N_c, N_r, N_q ＝ 土の内部摩擦角によって決定される支持力係数（表 10.7.2 参照）

表 10.7.2　支 持 力 係 数

ϕ	Nc	Nr	Nq
0°	5.3	0	3.0
5°	5.3	0	3.4
10°	5.3	0	3.9
15°	6.5	1.2	4.7
20°	7.9	2.0	5.9
25°	9.9	3.3	7.6
28°	11.4	4.4	9.1
32°	20.9	10.6	16.1
36°	42.2	30.5	33.6
40°以上	95.7	114.0	83.2

(b) 降伏引揚支持力 : q_{ty}

図 10.7.2に示すように、鉛直掘りで掘削，埋戻しを行った基礎の引揚支持力は、下記に示すせん断法により計算を行う。

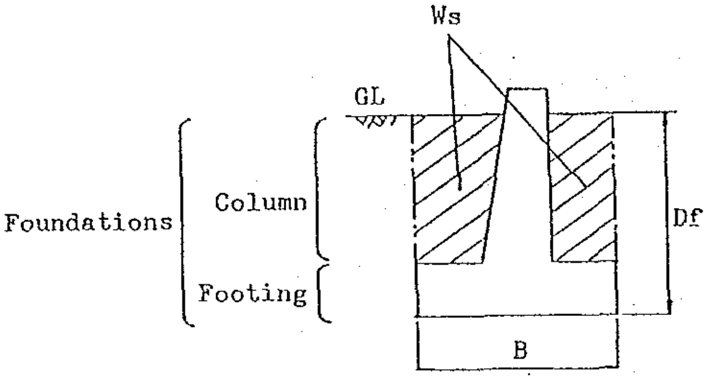


図 10.7.2

$$q_{ty} = K\left\{W_c + W_s + \frac{1}{1.5}\cdot L\cdot D_f\left(C + \frac{1}{2} r s_2\cdot Df\cdot \tan\phi/(1+\sin\phi)\right)\right\}$$

$$q_{ty} = K\left\{W_c + W_s + \frac{1}{1.5}\cdot L\cdot t\cdot C\right\}$$ (岩盤の場合)

ここに

K = 転倒モーメントによる引揚支持力の低減率

$$K = \frac{1}{1+6eB/(B^2+b^2)} \quad (\geqq 0.67)$$

B = 床 板 巾 (m)

e = $Q_B\cdot H/(T - W_{CT})$ (m)

b = 柱体底面巾 (m)

Q_B = 腹材からの水平力 (ton)

H = 柱体高さ (m)

T = 鉄塔から基礎体に働く引揚力 (ton)

W_{CT} = 柱体と柱体直下の基礎床板重量 (ton)

W_c = 基礎体重量 (ton)

(ただし、地下水位下は、水中重量を考慮する。)

W_s = 埋戻し土の重量（ton）

（ただし、地下水位下は、水中重量を考慮する。）

L = 床板の全辺長(m) − 4・B

D_f = 根入深さ（m）

C = 床板直上の土の平均粘着力（ton/m²）

r_{s2} = 床板上の土の平均単位体積重量（ton/m³）

（ただし、地下水位下は、水中重量を考慮する。）

ϕ = 床板上の土の内部摩擦角（度）

τ = せん断抵抗（ton/m²）

(c) 最大地盤反力；$\sigma cmax$

水平力Q_Bが圧縮力と同時に作用する場合の、床板縁端部における最大地盤反力（σcamx）は、下記に示す公式によって算出される。（図 10.7.3）

$$\sigma cax = \frac{P}{A} \cdot \mu$$

ここに

P = 基礎体に働く鉛直力（ton）

$$P = C + W_c + W_s$$

C = 圧 縮 力（ton）

W_c = 基礎体重量（ton）

W_s = 床板上の土の重量（ton）

A = 床板底面積（m²）

$$A = B \times B$$

μ = 水平力Q_Bによって生じる転倒モーメントによる圧縮反力の増加率

(i) $\mu = 1 + \frac{6e}{B}$；（$e \leqq \frac{B}{6}$ の場合）

(ii) $\mu = \frac{2}{3(1/2 - e/B)}$；（$\frac{B}{6} < e < \frac{B}{2}$ の場合）

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ここに

e ＝ Fig. 10.7.3 に示す偏心量 (m)

B ＝ 床板巾 (m)

H＋t ＝ 柱体天端より床板底面までの深さ (m)

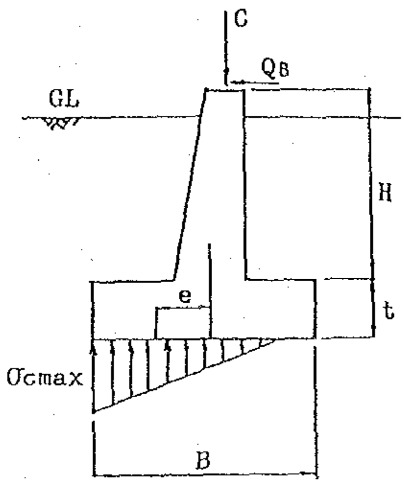

図 10.7.3

(d) 安全率

圧縮力，引揚力に対する安全率を下記の表 10.7.3に示す。

表 10.7.3　各荷重に対する安全率

荷重の種類	A	B	E
圧縮力	$\dfrac{q\,ct}{\sigma\,cmax} \geqq 2.0$	$\dfrac{q\,ct}{\sigma\,cmax} \geqq \dfrac{2.0}{1.5}$	—
引揚力	$\dfrac{q\,ty}{T} \geqq 2.0$	$\dfrac{q\,ty}{T} \geqq \dfrac{2.0}{1.5}$	$Wc+Ws \geqq To$

ここに、

T ＝ 引揚率 (ton)

To ＝ 固定引揚荷重 (E荷重 (ton))

表 10.7.3に示す安全率に於いて、安全率 2.0は、常時A荷重の降伏支持力に対する値である。短期荷重のB荷重に関しては、降伏支持力に対してではなく、その 1.5倍に当たる極限支持力に対して安全率 2.0を考える。(図 10.7.4)

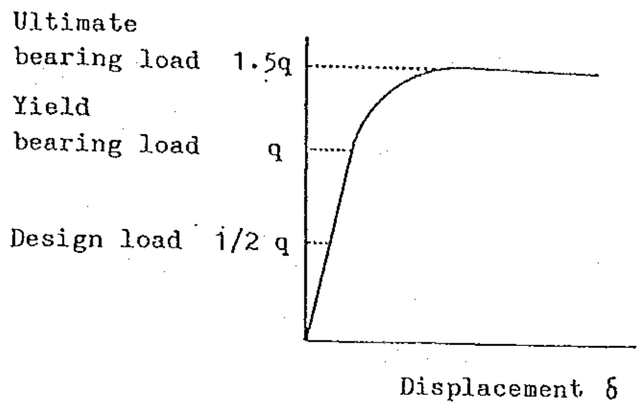

図 10.7.4

(e) イカリ材

イカリ材の形状を図 10.7.5に示す。

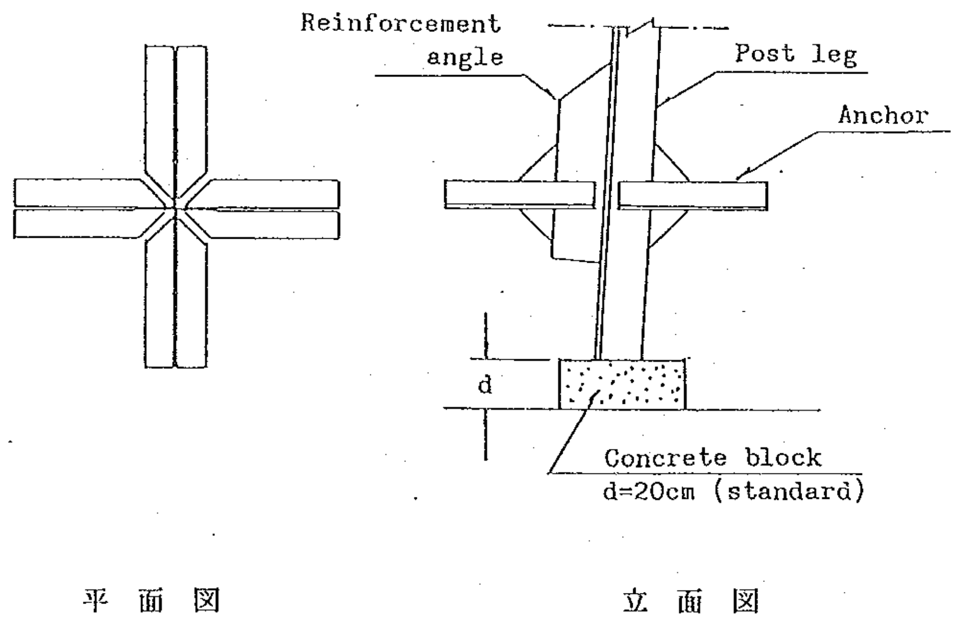

図 10.7.5　イカリ材の構造図

(3) 杭基礎の安定計算（III type）

杭基礎は、通常軟弱地盤の地域に用いられる。杭は、軸方向の圧縮力に対して、地中部の杭の表面に働くせん断抵抗力（周面摩擦力）で支持されるものと、杭先端で支持されるもの（先端支持力）、または、その両方によって支持されるものがある。

主に、周面摩擦力のみに支持される杭を摩擦杭と呼び、杭先端支持力および周面摩擦力の両方で支持される杭を支持杭と呼ぶ。

引揚力に対しては、周面摩擦力だけで支持される。

(a) 圧縮支持力

（i） 摩 擦 杭

$$Qcy = \frac{1}{1.5}\{(Ns \cdot ls/5 + qu \cdot lc/2) \cdot U\} - 2 \cdot Wp$$

（ii） 支 持 杭

○鋼管打込杭

$$Qcy = \frac{1}{1.5}\{(30\eta \cdot N \cdot Ap + (Ns \cdot ls/5 + qu \cdot lc/2) \cdot U\} - 2 \cdot Wp$$

○場所打杭

$$Qcy = \frac{1}{1.5}\{(15 \cdot N \cdot Ap + (Ns \cdot ls/5 + qu \cdot lc/2) \cdot U\} - 2 \cdot Wp$$

ここに

Qcy = 降伏圧縮支持力（ton/本）

η = 杭の閉そく効率

$\eta = 0.16 \cdot L_B/D \ (\leqq 0.8)$ （開端杭）

$\eta = 1.0$ （閉端杭）

L_B = 支持層への貫入長さ（m）

D = 杭の内径（m）

N = 支持層の平均N値

Ap = 杭先端断面積（m^2）

Ns = 砂質土層の平均N値

ls = 砂質土層の層厚（m）

qu = 粘性土層の平均一軸圧縮強度（≦ 16 t/m^2）（ton/m^2）

ただし、試験データがない場合のquの指定式を以下に示す。

$qu = 1.25 \cdot Nc$

lc = 粘性土層の層厚（m）

U = 杭の周長（m）

Wp = 杭の重量（ton/本）

(b) 引揚げ支持力

$$Qcy = \frac{1}{1.5}\{(Ns \cdot ls/5 + qu \cdot lc/2) \cdot U\} + Wp'$$

ここに

Qty ＝ 降伏引揚支持力 (ton/本)

Ns ＝ 砂質土層の平均N値

ls ＝ 砂質土層の層厚 (m)

qu ＝ 粘性土層の平均一軸圧縮強度 ($\leqq 6\ t/m^2$) (ton/m^2)

lc ＝ 粘性土層の層厚 (m)

U ＝ 杭の周長 (m)

Wp' ＝ 水中重量を考慮した杭の重量 (ton/本)

(c) 杭の圧縮反力

$$Ncmax = \frac{P}{n} + \frac{M}{\Sigma m X i^2} \cdot Xo$$

ここに

$Ncmax$ ＝ 杭の最大地盤反力 (ton/本)

P ＝ 鉛直方向の合力 (ton)

$P = C + Wc + Ws$

ここに

C ＝ 鉄塔からの圧縮力 (ton)

Wc ＝ 基礎体重量 (ton)

Ws ＝ 床板直上の土の重量 (ton)

M ＝ 床板図心における転倒モーメント (t.m)

$M = Q_B \cdot (H + t)$

ここに

Q_B ＝ 腹材応力の水平成分 (ton)

H ＝ 柱体高さ (m)

t ＝ 床板厚さ (m)

Xo ＝ 杭群重心から最外側杭の中心までの距離 (m)

$\Sigma m \cdot X i^2$ ＝ 杭群の重心に関する二次モーメント (m^2)

n ＝ 杭本数 (本)

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(d) 杭の引揚反力

$$N_{tmax} = (1-K)\left(\frac{T}{n} + \frac{M}{\Sigma m \cdot X i^2} \cdot X_o\right)$$

ここに

N_{tmax} ＝ 杭の最大引揚反力（ton/本）

K ＝ 基礎体の荷重分担率

$$K = \frac{Wc' + Ws'}{Wc' + Ws' + n \cdot Q_{ty}}$$

Wc' ＝ 浮力を考慮した基礎体重量（ton）

n ＝ 杭本数（本）

Q_{ty} ＝ 降伏引揚支持力（ton/本）

T ＝ 鉄塔からの引揚力（ton）

M ＝ 床板図心における転倒モーメント（t･m）

X_o ＝ 杭群重心から最外側杭の中心までの距離（m）

$\Sigma m \cdot X i_2$ ＝ 杭群重心に関する二次モーメント（m^2）

(e) 安 全 率

圧縮力、引揚力に対する安全率を下表 10.7.4に示す。

表 10.7.4　各荷重に対する安全率

荷重の種類	A	B	E
圧 縮 力	$\frac{q_{ct}}{N_{cmax}} \geqq 2.0$	$\frac{q_{ct}}{N_{cmax}} \geqq \frac{2.0}{1.5}$	—
引 揚 力	$\frac{q_{ty}}{N_{tmax}} \geqq 2.0$	$\frac{q_{ty}}{N_{tmax}} \geqq \frac{2.0}{1.5}$	$W_c + W_s + n W_p' \geqq T_o$

ここに、

T_o ＝ 固定引揚荷重（E荷重（ton））

(4) 設計条件

(a) 許容応力度

表 10.7.5

項目		許容応力度 長期	許容応力度 短期	備考
鉄筋	引張応力度	2,000 kg/cm²	3,000 kg/cm²	SD30
コンクリート	圧縮応力度	60 kg/cm²	90 kg/cm²	σ28= 180 kg/cm²
	せん断応力度	6 kg/cm²	9 kg/cm²	
	支圧応力度	60 kg/cm²	90 kg/cm²	
	付着応力度	12.0 kg/cm²	18.0 kg/cm²	
鋼管杭	圧縮応力度	1,600 kg/cm²	2,400 kg/cm²	SKK 41
	引張応力度	1,600 kg/cm²	2,400 kg/cm²	
	せん断応力度	900 kg/cm²	1,350 kg/cm²	
弾性係数	コンクリート	2.4x10⁵ kg/cm²		σ28= 180 kg/cm²
	鋼管杭	2.4x10⁵ kg/cm²		SKK 41

o 鉄筋コンクリートの許容応力度は、JEC-127 送電用鉄塔設計標準により計算している。(日本電気学会)

鉄筋の弾性係数と、鋼管杭の弾性係数は、それぞれ建築基礎構造設計基準と建築鋼ぐい基礎設計施工規準(日本建築学会)を参考に算出した。

(b) 単位重量

表 10.7.6 材料の単位重量

	空中(t/m³)	水中(t/m³)	備考
鉄筋コンクリート	2.4	1.4	σ28=180kg/cm²
鋼管杭	7.85	6.85	SKK 41

/ヾ/

本報告書の TLG-1-145 ページから TLG-1-160 ページまでは、英文報告書のページ番号と合わせるために欠番とする。

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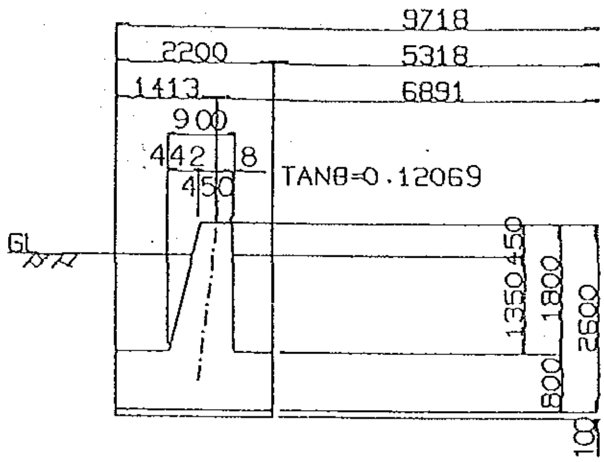
10．7．3 基礎設計計算

(1). As-D-I

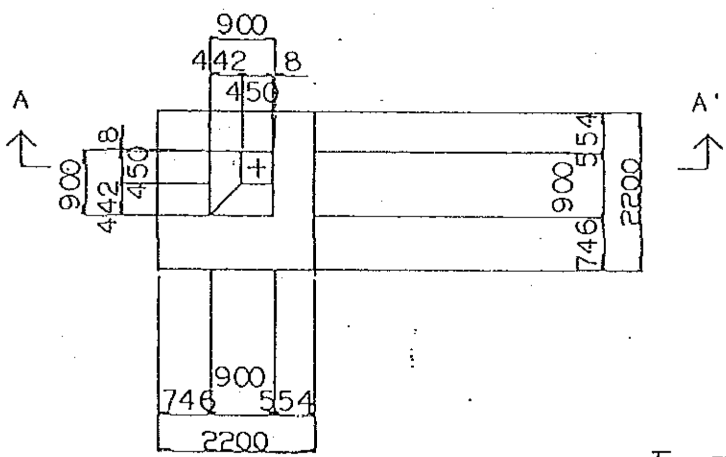
$C = 37.54^t$
$T = 25.49$
$Q = 2.89$
$Q_B = 1.10$

PROFILE

S=1/100



A-A' SECTION



$F_c = 7.21 > 2.00$

$F_t = 2.07 > 2.00$

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

------------------------------INPUT DATA--

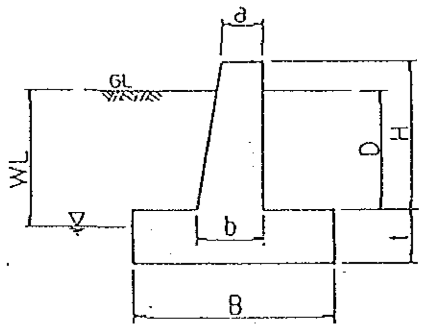
TOWER NAME............NO. AS-D-I

(1)LOAD CONDITION

Comp. Load...................C = 37.54 TON
Uplift Load..................T = 25.49 TON
Hori. Load...................Q = 2.89 TON
Hori. Load...................QB= 1.10 TON
E-Load.......................TO= 14.24 TON

(2)DIMENSION OF FOUNDATION

a = 0.450 M
b = 0.900 M
B = 2.200 M
t = 0.800 M
H = 1.800 M
D = 1.350 M
WL=- 10.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation).................C1= 0.000 TON/M^2
Cohesion (on the foundation)......................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation)....φ = 40.000 °
Bearing Capacity Factor..N1= 95.400 N2=114.000 N3= 83.200 (φ=40 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 1

-------------------------------RESULT-------------------------------

(5)VERTICAL LOAD

```
Weight of concrite..........................WC=   11.334 TON
Weight of Soil..............................WS=   10.439 TON
Weight of Column and a part under Column...WC'=    3.596 TON
Vertical Load.......................P=C+WC+WS=   59.313 TON
```

(6)PILE CAPACITY

```
Comp. Side.................................QCY= 100.000 TON/M^2
Uplift Side................................QTY=  51.451 TON/M^2
Hori. Side.................................QHY=  40.000 TON/M^2
```

(7)SOIL REACTION   Upper Limit

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.048 M <= B/6=  0.367 M
Incremental Rate...................MU=1+6*E/B=  1.132
Max. Soil Reaction................Qmax=P/A*MU= 13.866 TON/M^2
```

2)Uplift Side

```
Eccentricity....................E=QB*H/(T-WC')=  0.090 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.826
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  1.642 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side.......................SFC=QCY/Qmax=  7.211 >=  2.00 =OK=
Uplift Side.........................SFT=QTY/T=  2.018 >=  2.00 =OK=
Hori. Side.....................SFH=QHY/QHmax= 24.360 >=  2.00 =OK=
```

(10)E-LOAD   From Upper Limit.

```
WC+WS'=   19.685 TON >= TO=   14.240 TON =OK=
   (WS'=WS*0.8)
```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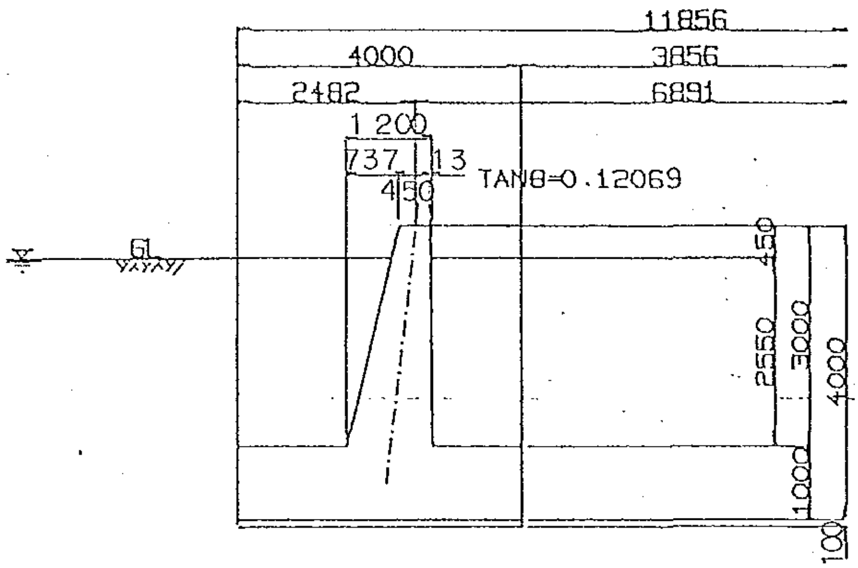
(2). As- C -I

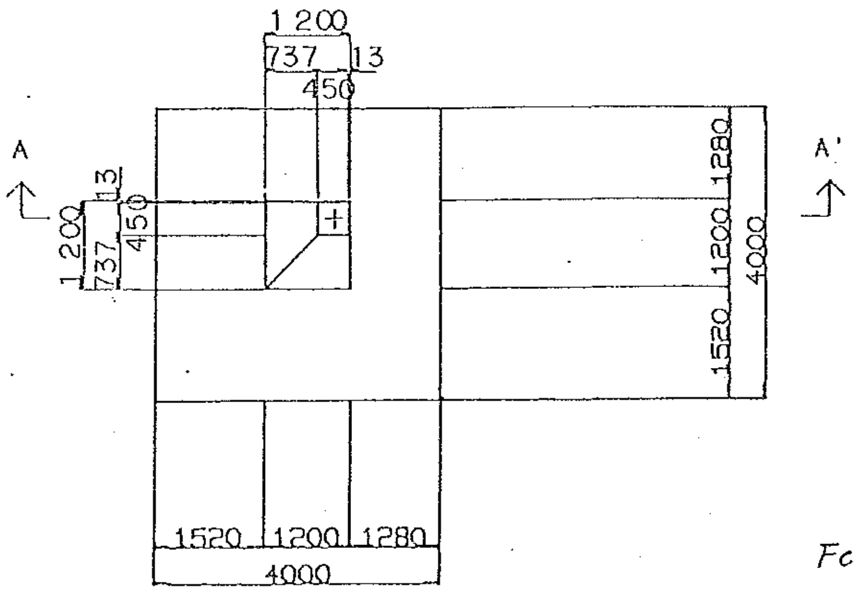
C= 37.54^t
T= 25.49
Q= 2.89
Q= 1.10

PROFILE

S=1/100



A - A' SECTION



Fc= 7.37 > 2.0
Ft= 2.13 > 2.0

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

------------------------------INPUT DATA------------------------------

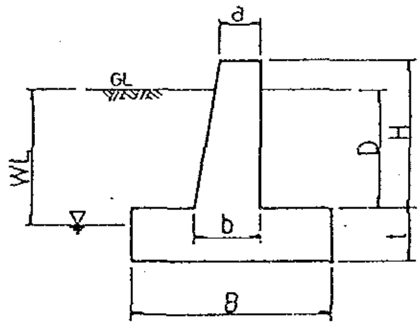
TOWER NAME............NO. A_5 -C-I

(1)LOAD CONDITION

Comp. Load....................C = 37.54 TON
Uplift Load...................T = 25.49 TON
Hori. Load....................Q = 2.89 TON
Hori. Load....................QB= 1.10 TON
E-Load........................TO= 14.24 TON

(2)DIMENSION OF FOUNDATION

a = 0.450 M
b = 1.200 M
B = 4.000 M
t = 1.000 M
H = 3.000 M
D = 2.550 M
WL=- 0.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation)................C1= 0.000 TON/M^2
Cohesion (on the foundation)......................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation)....φ = 35.000 °
Bearing Capacity Factor..N1= 35.400 N2= 23.400 N3= 27.800 (φ =35 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

-----------------------------------RESULT---

(5)VERTICAL LOAD

```
Weight of concrete.........................WC=   43.638 TON
Weight of Soil.............................WS=   69.720 TON
Weight of Column and a part under Column...WC'=   5.187 TON
Vertical Load.......................P=C+WC+WS=  150.898 TON
```

(6)PILE CAPACITY

```
Comp. Side................................QCY=  72.603 TON/M^2
Uplift Side...............................QTY=  54.378 TON/M^2
Hori. Side................................QHY=  29.041 TON/M^2
```

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.029 M <= B/6=  0.667 M
Incremental Rate...................MU=1+6*E/B=  1.044
Max. Soil Reaction................Qmax=P/A*MU=  9.844 TON/M^2
```

2)Uplift Side

```
Eccentricity..................E=QB*H/(T-WC')=  0.163 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.817
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  0.723 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side.......................SFC=QCY/Qmax=  7.375 >=  2.00 =OK=
Uplift Side........................SFT=QTY/T=  2.133 >=  2.00 =OK=
Hori. Side......................SFH=QHY/QHmax= 40.195 >=  2.00 =OK=
```

(10)E-LOAD

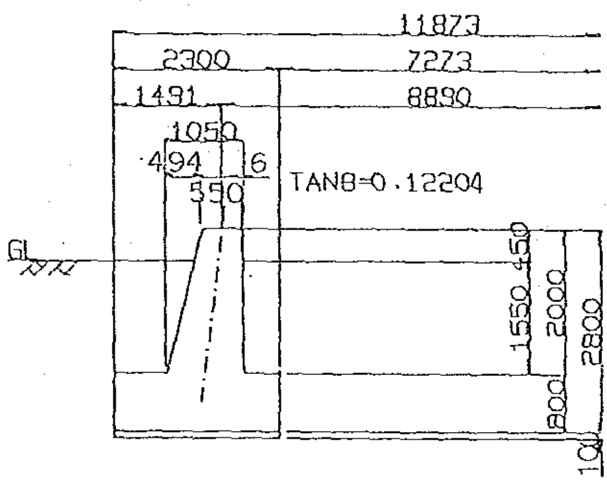
```
WC+WS'=   42.614 TON >= TO=   14.240 TON =OK=
   (WS'=WS*0.8)
```

(3). A-D-I

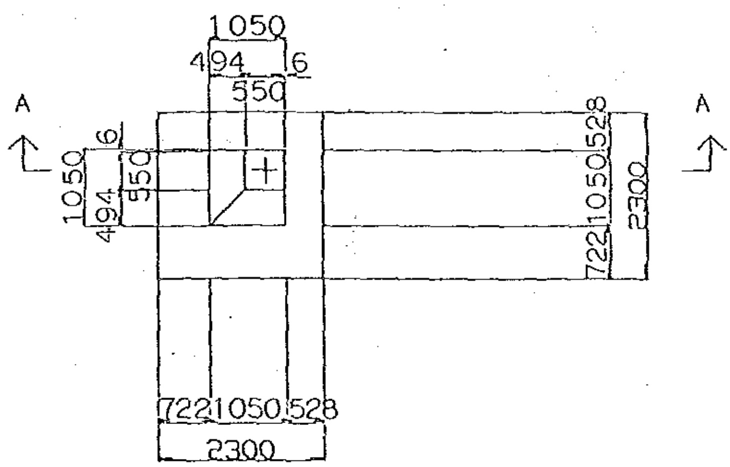
$C = 49.27^{t}$
$T = 34.09$
$Q = 4.14$
$Q_{A} = 1.53$

PROFILE

S=1/100



A-A' SECTION



$F_c = 6.11 > 2.00$

$F_t = 2.01 > 2.00$

PLAN

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INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

-------------------------------INPUT DATA-------------------------------------

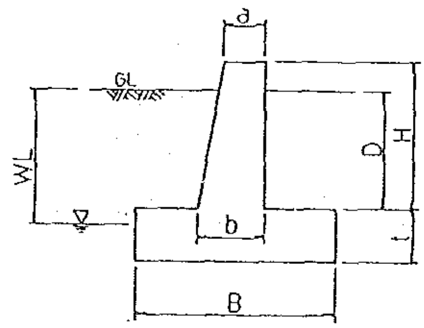
TOWER NAME............NO. A-D-I

(1)LOAD CONDITION

Comp. Load...................C = 49.27 TON
Uplift Load..................T = 34.09 TON
Hori. Load...................Q = 4.14 TON
Hori. Load...................QB= 1.53 TON
E-Load.......................TO= 1.89 TON

(2)DIMENSION OF FOUNDATION

a = 0.550 M
b = 1.050 M
B = 2.300 M
t = 0.800 M
H = 2.000 M
D = 1.550 M
WL=- 10.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation).................C1= 0.000 TON/M^2
Cohesion (on the foundation)......................C2= 0.000 TON/M^
Angle of Internal Friction (on the foundation)....φ = 40.000 °
Bearing Capacity Factor..N1= 95.700 N2=114.000 N3= 83.200 (φ =40 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

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-------------------------------RESULT-------------------------------

(5)VERTICAL LOAD

```
Weight of concrite.........................WC=   13.329 TON
Weight of Soil.............................WS=   12.679 TON
Weight of Column and a part under Column...WC'=   5.289 TON
Vertical Load.......................P=C+WC+WS=   75.278 TON
```

(6)PILE CAPACITY

```
Comp. Side................................QCY= 100.000 TON/M^2
Uplift Side...............................QTY=  68.558 TON/M^2
Hori. Side................................QHY=  40.000 TON/M^2
```

(7)SOIL REACTION   Upper Limit

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.057 M <= B/6=  0.383 M
Incremental Rate..................MU=1+6*E/B=  1.148
Max. Soil Reaction.............Qmax=P/A*MU=  16.343 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=  0.106 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.813
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*l)=  2.250 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side......................SFC=QCY/Qmax=  6.118 >=  1.33 =OK=
Uplift Side........................SFT=QTY/T=  2.011 >=  1.33 =OK=
Hori. Side....................SFH=QHY/QHmax= 17.777 >=  1.33 =OK=
```

(10)E-LOAD   From Upper Limit

```
WC+WS'=   23.472 TON >= TO=    1.890 TON =OK=
   (WS'=WS*0.8)
```


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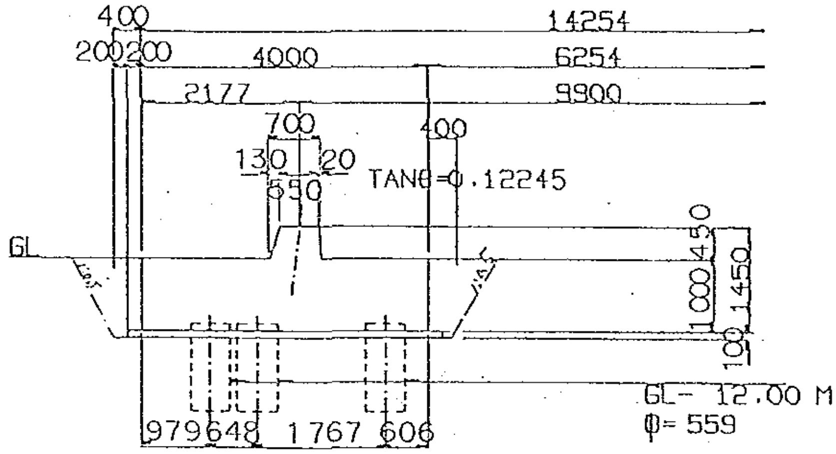
(4). AL-S-III

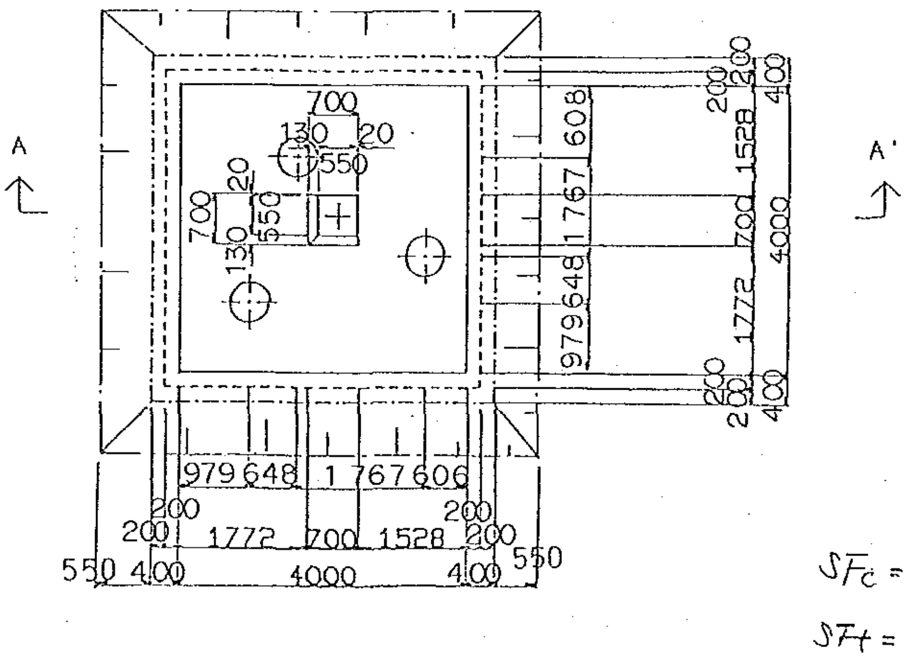
$C = 57.95^{t}$
$T = 40.17$
$Q_A = 1.72$

PROFILE

S=1/100



A-A' SECTION



$SF_c = 2.54 > 2.00$

$SF_t = 2.01 > 2.00$

PLAN

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PILE TYPE TOWER FOUNDATION (Ⅲ)

-------------------------------INPUT DATA-------------------------------

TOWER NAME.....NO. AL-S-Ⅲ

(1)LOAD CONDITION

Comp. Load.........................C = 57.95 TON
Uplift Load........................T = 40.17 TON
Hori. Load.........................Q = 6.89 TON
Hori. Load.........................QB= 1.72 TON
E-Load.............................TO= 2.29 TON

(2)STRATUM DATA

No.	Stratum No.	N-Value(N)	Thickness(L)	N*L
1	1	0(0)	1.00 M	0.000(0.000)
2	1	1(1)	2.00 M	2.000(2.000)
3	2	3(3)	1.00 M	2.500(2.500)
4	1	7(7)	6.50 M	45.500(45.500)
5	2	16(6)	1.50 M	24.000(9.000)

Sand (total)...................NS*LS= 47.500 TON
(47.500)
Clay (total)...................QU*LC= 26.500 TON
(11.500)

(3)WATER TABLE......................WL=- 0.000

(4)N-VALUE OF BEARING STRATUM.........N = 29

(5)BULK DENSITY

Concrete...........................γ = 2.400 TON/M^3
Soil (on the foundation)...........S2= 0.000 TON/M^3

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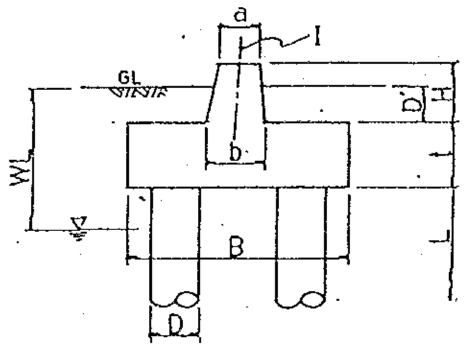
(6)DIMENSION OF FOUNDATION

a = 0.550 M
b = 0.700 M
B = 4.000 M
t = 1.000 M
H = 0.450 M
D'= 0.000 M
I = 0.12245



(7)PILE TYPE (driven pile...1,cast in place conc. pile...2).... 2

(8)PILE CONDITION

Diameter of Pile....................D = 0.559 M
Length of Pile......................L = 12.000 M
Unit Weight (without buoyancy)......G1= 0.589 TON/M
Unit Weight (with buoyancy).........G2= 0.344 TON/M
Blockade Ratio......................R = 1.000

(9)LIMIT OF PILE CAPACITY

Comp. Side........................QCA= 124.000 TON/UNIT
Uplift Side.......................QTA= 49.000 TON/UNIT

(10)CONFIGURATION

Row of pile	Number(UNIT)	Distance(M)
1	1	1.394
2	1	-0.373
3	1	-1.021

Eccentricity.......................E= 0.000 M

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------------------------------------RESULT------------------------------------

(11)VERTICAL LOAD

1)Without Buoyancy

```
Weight of Concrete...................WC=    38.824 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=    96.774 TON
```

2)With Buoyancy

```
Weight of Concrete...................WC=    22.824 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=    80.774 TON
```

(12)PILE CAPACITY

```
Comp. Side...QCY=15*R*N*π*D^2/4+((NS*LS)/5+(QU*LC)/2)*π*D)/1.5-2*WP
               =  84.849 TON/UNIT

Uplift Side..QTY=((NS*LS)/5+(QU*LC)/2)*π*D+1.5WP')/1.5
               =  21.638 TON/UNIT
```

(13)OVERTURNING MOMENT

1)Without Buoyancy

```
Comp. Side..............MC=QB*(H+t)+P*E=     2.494 TON-M
Uplift Side....MT=QB*(H+t)+ | T-WC-WS | *E=     2.494 TON-M
```

2)With Buoyancy

```
Comp. Side..............MC=QB*(H+t)+P*E=     2.494 TON-M
Uplift Side....MT=QB*(H+t)+ | T-WC-WS | *E=     2.494 TON-M
```

(14)WEIGHT OF PILE

```
Without Buoyancy.....................WP=     6.479 TON/UNIT
With Buoyancy.......................WP'=     3.784 TON/UNIT
```

(15)LOAD DISTRIBUTIVE

Without Buoyancy......................KA= 0.374
With Bouyancy.........................KA= 0.260

(16)PILE REACTION

1)Comp. Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	33.371	28.037
2	31.960	26.627
3	31.443	26.110

2)Uplift Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	9.075	10.730
2	8.193	9.687
3	7.869	9.304

(17)SAFETY FACTOR

1)Without Buoyancy

Comp. Side.................SFC=QCY/NCmax= 2.542 >= 2.0 =OK=
Uplift Side................SFT=QTY/NTmax= 2.384 >= 2.0 =OK=

2)With Buoyancy

Comp. Side.................SFC=QCY/NCmax= 3.026 >= 2.0 =OK=
Uplift Side................SFT=QTY/NTmax= 2.016 >= 2.0 =OK=

(18)E-LOAD

1)Without Buoyancy

WC+WS+N*WP= 58.261 TON >TO= 2.290 TON =OK=

2)With Buoyancy

WC+WS+N*WP'= 34.176 TON >TO= 2.290 TON =OK=

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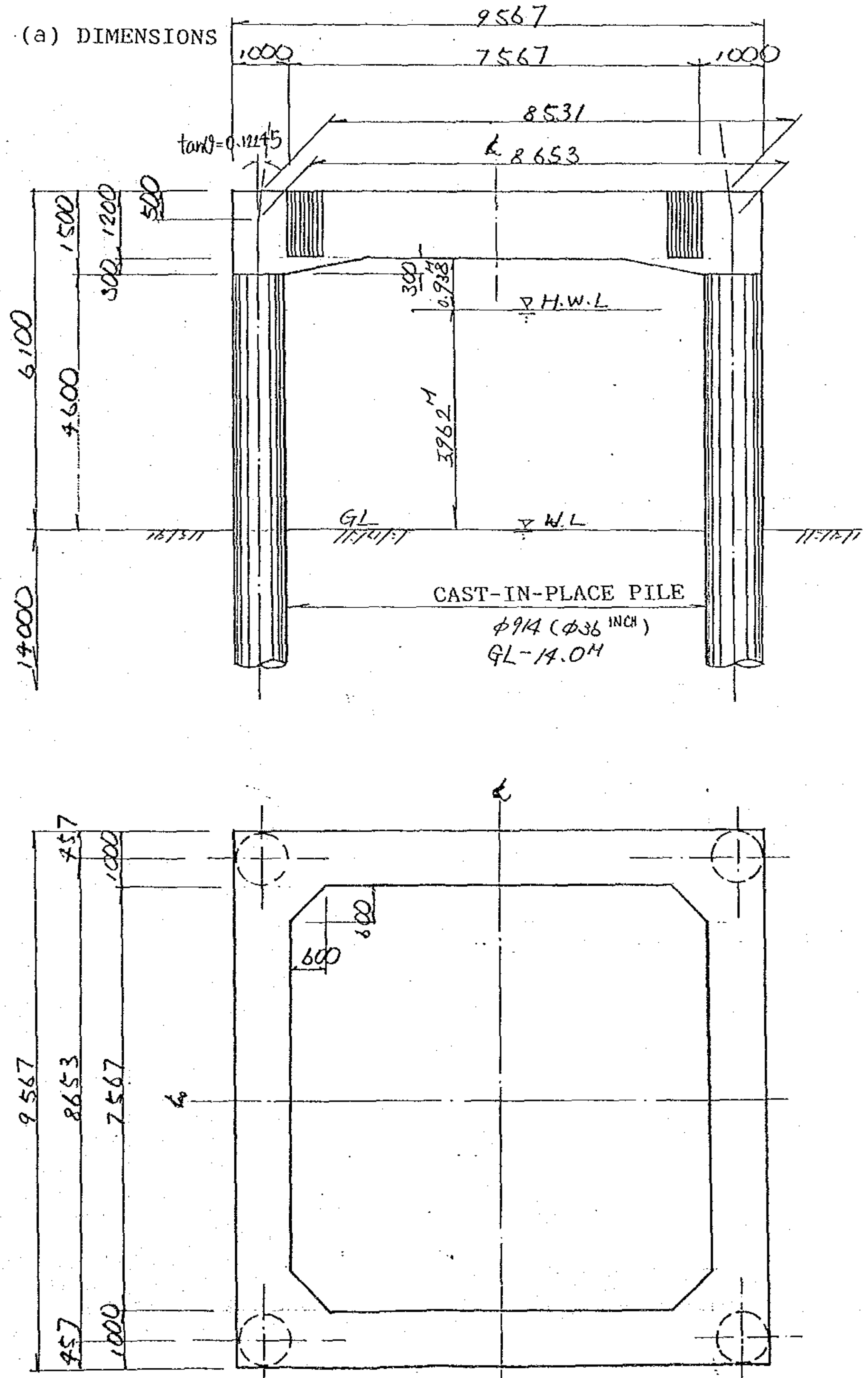
(5). AL-R

(a) DIMENSIONS



(b) Computation of Bearing Capacity of Ground

KESC WEST WHARF - Transmission Line.

BORE HOLE NO: TL-1

BORE LOG

Date: 28.5.89 to 30.5.89 Ground Elev: 3.066m Ground Water Table: 3.00m

DEPTH (m)	THICKNESS (m)	SOIL NAME/DESCRIPTION
3.00	3.00	Brown, very loose, fine SAND.
4.00	1.00	Dark brown, decayed, soft wooden pieces.
10.50	6.50	Grey, loose, fine SAND with traces of shell fragments.
12.50	2.00	Grey, hard, Silty CLAY.
14.50	2.00	Grey, very dense, fine SAND.
22.50	8.00	Brown, very dense, fine to medium, SAND with traces of coarse SAND.
24.50	2.00	Brownish grey, hard, Silty CLAY.
30.50	6.00	Brownish grey, fine to medium, very dense SAND with traces of coarse sand and gravel.
		Borehole completed.

STANDARD PENETRATION TEST Blows/foot (N-Value): 20 40 60 80 100

	2
	1
	2
	2
	11
	7
	5
	4
	8
	8
	78
	84
	83
	68
	65
	89
	107
	110
	120
	130

Ø36 INCH

GL-14.0 M

SPT Sample: □

NB: Ground Water Table reported in all Borecharts indicate depth of water below the existing ground level.

PGEL
PENCON GEO-ENGINEERING (PVT.) LTD.
9 Sunny Side Road, Civil Lines,
P.O. Box No: 3969, KARACHI-4

No.	Stratum No.	N-Value(N)	Thickness(L)	N*L
1	1	1(1)	3.00 M	3.000(3.000)
2	2	3(3)	1.00 M	2.500(2.500)
3	1	7(7)	6.50 M	45.500(45.500)
4	2	16(6)	2.00 M	32.000(12.000)
5	1	50(50)	1.50 M	75.000(75.000)

Sand (total)....................NS*LS= 123.500 TON
(123.500)
Clay (total)....................QU*LC= 34.500 TON
(14.500)

WATER TABLE.......................WL=- 0.000

N-VALUE OF BEARING STRATUM.........N = 46

PILE CONDITION

Diameter of Pile...................D = 0.914 M
Length of Pile.....................L = 14.000 M
Unit Weight (without buoyancy)......G1= 1.575 TON/M
Unit Weight (with buoyancy).........G2= 0.919 TON/M
Blockade Ratio.....................R = 1.000

LIMIT OF PILE CAPACITY

Comp. Side.......................QCA= 334.000 TON/UNIT
Uplift Side......................QTA= 131.000 TON/UNIT

PILE CAPACITY

Comp. Side...QCY=15*R*N*π*D^2/4+((NS*LS)/5+(QU*LC)/2)*π*D)/1.5-2*WP
= 334.000 TON/UNIT — 330 t

Uplift Side..QTY=((NS*LS)/5+(QU*LC)/2)*π*D+1.5WP')/1.5
= 74.027 TON/UNIT — 74 t

WEIGHT OF PILE

Without Buoyancy.....................WP= 22.050 TON/UNIT
With Buoyancy........................WP'= 12.866 TON/UNIT

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15P

(c) Coefficient of Lateral Subgrade Reaction

The coefficient of lateral subgrade reaction should be calculated using the following formula, dividing the ground into five layers.

$$ko = \alpha EoD^{-3/4}$$

where,

ko : coefficient of lateral subgrade reaction (kg/cm³)

α : coefficient for computation of Eo (α = 0.2) (Eo is estimated using the N-value of 28N.)

Eo : deformation modulus of ground (kg/cm²)

D : effective pile diameter (91.4 cm)

Types / Stratigraphy	α	E_0 (kg/cm²)	D (cm)	k_0 (kg/cm³)	k (kg/cm²)	Remarks
Fine sand	0.2	28	91.4	0.189	17.3	
Humus	〃	56	〃	0.379	34.6	
Fine sand	〃	196	〃	1.326	121.2	
Silty clay	〃	1400	〃	9.472	865.7	
Fine sand	〃	1400	〃	9.472	865.7	

Note : k = ko·D

(d) Axial Spring Constant of Pile

The axial spring constant of the pile should be computed using the following formula.

$$kv = a \cdot \frac{Ap \cdot Ep}{\ell}$$

where,

kv : axial spring constant of pile (kg/cm)

Ap : net sectional area of pile (cm^2)

Ep : elastic modulus of pile body (kg/cm^2)
(= 270,000 kg/cm^2)

ℓ : pile length (cm)

a : $a = 0.022(\ell/D) - 0.05$
(cast-in-place pile).

$$Kv = (0.022 \times \frac{1400}{91.4} - 0.05) \times \frac{91.4^2 \times \pi/4 \times 270000}{1400}$$

$$= 363\,137 \ ^{kg/cm}$$

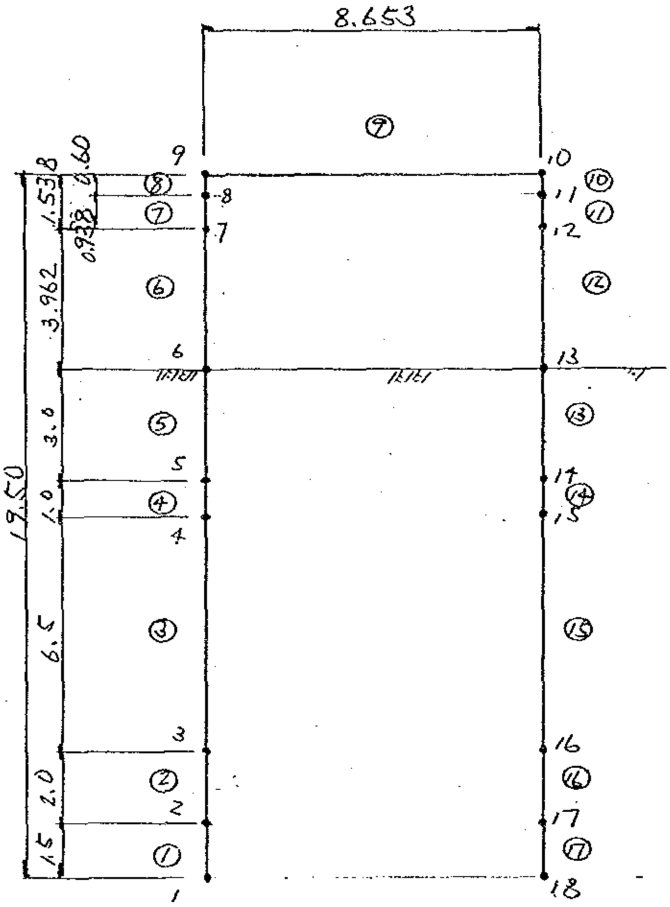
$$= 36300 \ ^{t/m}$$

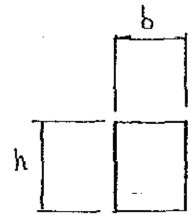
(e) Load Combination

Types \ Cases		Stationary	Abnormal	Oblique Wind	Oblique Wind	Earthquakes	Floods	E Load
Design Load		○	○	○	○	○	○	○
Water Flow Pressure							○	
Temperature Changes					○			
Wind Pressure		○	○	○	○		○	
Dead Weight of Tower						○		
Design Load	C	55.06	43.68	57.95	57.95	= E load	= Stationary	19.29
	T	37.28	31.77	40.17	40.17			2.29
	Q	11.28	40.17	6.89	6.89			2.05
Comparison Cases				※	※	※	※	※
Additional Rate of Stress	Foundation Body	1.0	1.5	1.0	1.15	1.5	1.5	1.0
	Pile Body	1.0	1.5	1.0	1.0	1.5	1.5	1.0
Remarks				CASE 1~3	CASE 13 ~15	CASE 4~6	CASE 7~9	CASE 10~12

No.

(f) Skeleton Diagram



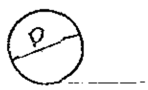


$$A = b \cdot h$$

$$= 1.00 \times 1.20 = 1.20^{m^2}$$

$$I = \frac{b \cdot h^3}{12}$$

$$= \frac{1.0 \times 1.2^3}{12} = 0.1440^{m^4}$$



$$A = \frac{\pi \cdot D^2}{4}$$

$$= \frac{\pi \times 0.914^2}{4} = 0.656^{m^2}$$

$$I = \frac{\pi \cdot D^4}{64}$$

$$= \frac{\pi \times 0.914^4}{64} = 0.0343^{m^4}$$

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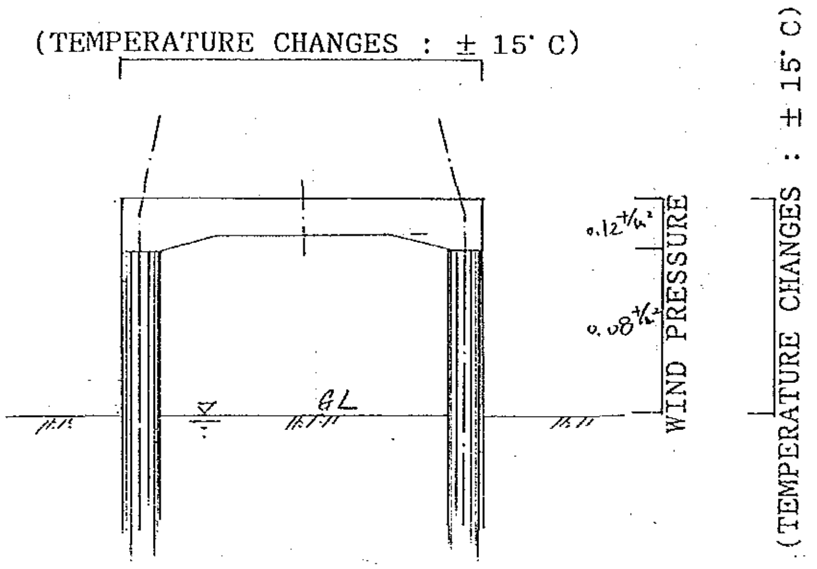
(g) Oblique Wind (taking account of temperature changes)

Load Computation



o Design Loads

$C = 57.95^t$

$T = 40.17$ "

$Q = 5.032 + 1.852 = 6.89^t$

o Dead Weight of Foundation

$w_1 = 1.00 \times 1.20 \times 2.4^{t/m^3} = 2.88^{t/m}$

$p_1 = (1/2 \times 9.567 \times 1.20 \times 1.00 + 1.00^2 \times 0.30) \times 2.4 = 14.50^t$

$p_2 = 1/2 \times 0.30 \times 1.50 \times 1.00 \times 2 \times 2.4 = 1.08$ "

$p_3 = 1/2 \times 0.60^2 \times 1.20 \times 2.4 = 0.52$ "

$P_1 = 16.10^t$

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o Wind Pressure

$$P_{H1} = \{ 1/2 \times 9.567 \times 1.20 + 1/2 \times (1.00 + 2.50) \times 0.30 \} \times 0.12^{t/m^2}$$
$$= 0.75^{t}$$

$$q_{H1} = 0.914 \times 0.08^{t/m^2} \qquad = 0.07^{t/m}.$$

o Without Water Flow Pressure

o Dead Weight of Pile

In the air $\quad q_1 = \pi/4 \times 0.914^2 \times 2.4^{t/m^3} \quad = 1.57^{t/m}$

In the water $\quad q_2 = \quad " \quad 1.4 \,'' \quad = 0.92 \,''$

(Temperature changes)

± 15°C

(Coefficient of linear expansion)

Concrete $\quad 1.0 \times 10^{-5} = EPS$

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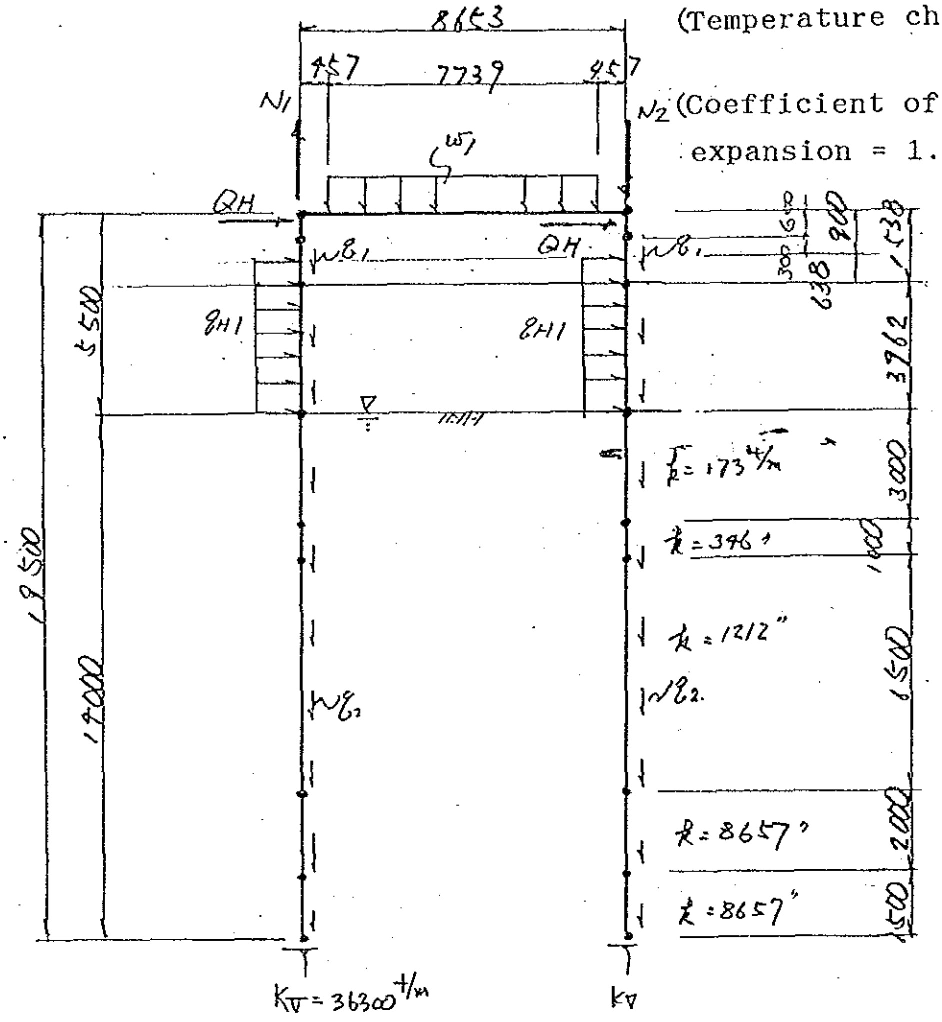
Load Diagram (at oblique wind : C-T) CASE 1
(CASE 13)
Temperature changes

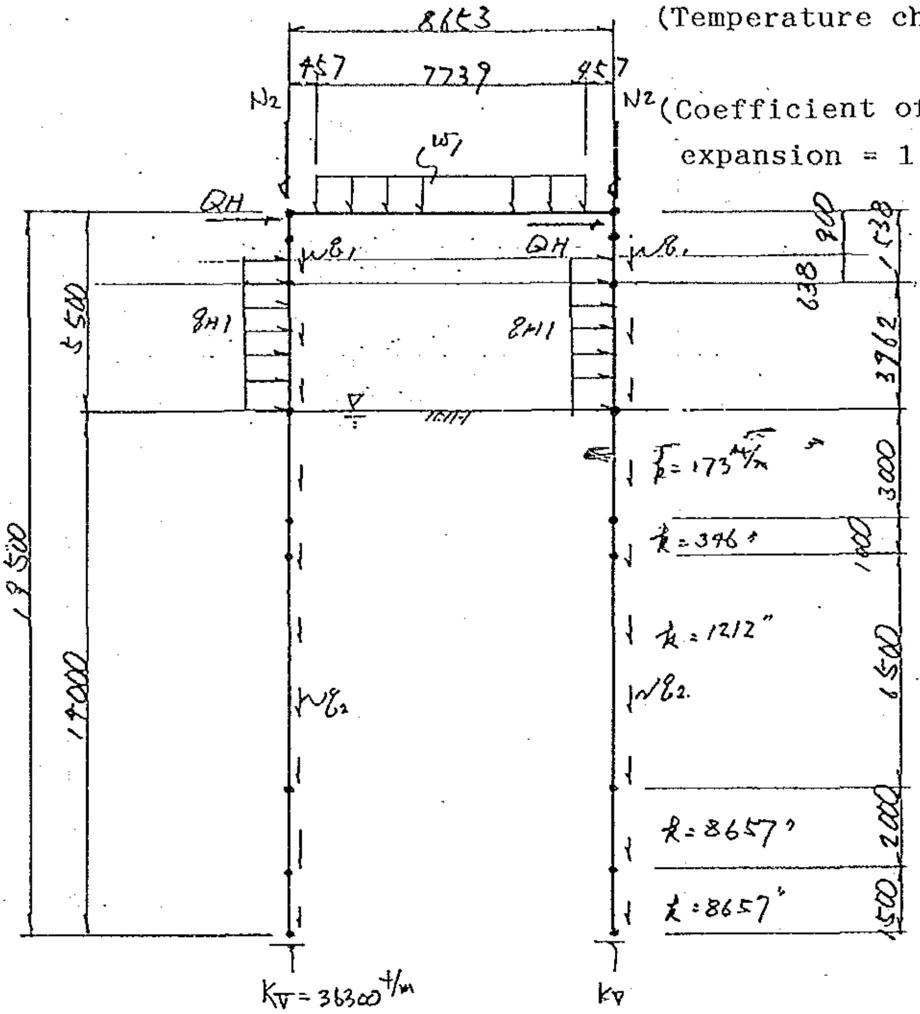
(Temperature changes : ± 15°C)
(Coefficient of linear expansion = 1.0 × 10^{-5})



$N_1 = P_1 - T = 16.10 - 40.17$
$= -24.07^t$
$N_2 = P_1 + C = 16.10 + 57.95$
$= 74.05^t$
$Q_H = Q + P_{H1} = 6.89 + 0.75$
$= 7.64^t$

Load Diagram (at oblique wind : C-C) CASE 2
(CASE 14)
Temperature changes

(Temperature changes : ± 15°C)
(Coefficient of linear expansion = 1.0 × 10^{-5})



$N_2 = 74.05^t$

$Q_H = Q + P_{H1} = 6.89 + 0.75$
$= 7.64^t$

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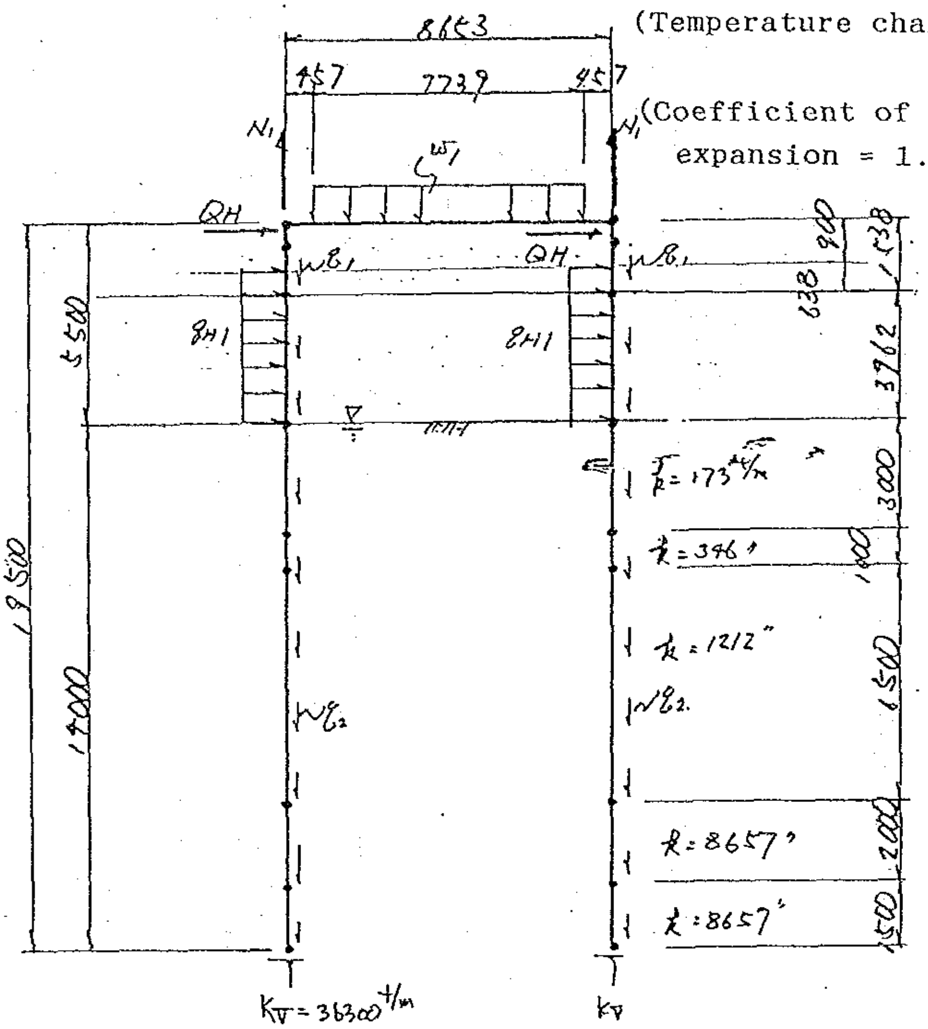
Load Diagram (at oblique wind : T-T) CASE 3
(CASE 15)
Temperature changes



(Temperature changes : ± 15° C)

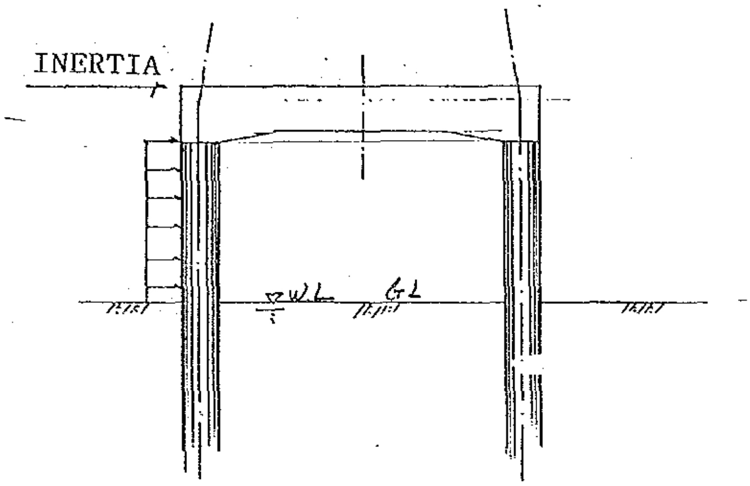
(Coefficient of linear expansion = 1.0 × 10^{-5})

$N_1 = -24.07$

$Q_H = Q + P_{H1} = 6.89 + 0.75$
$= 7.64^t$

(h) During Earthquakes

Load Computation



o Design Loads

$C = 19.29^{t}$

$T = 22.82^{t}$

$Q = 1.341 + 0.704 = 2.05^{t}$

o Dead Weight of Foundation

$w_1 = 2.88\ ^{t}/_{m}$

$P_1 = 14.50^{t}$ (Beam)

$P_2 = 1.08$ (Haunch)

$P_3 = 0.52$ (Horizontal haunch)

$P = 16.10^{T}$

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(19)

o Wind and Water Flow Pressures

Not taken into account

o Lateral Seismic Load

Lateral seismic coefficient $k_H = 0.1$

Beam	$(1/2 \times 9.567 \times 1.00 \times 1.20 \times 2.4^{t/m^3} + 1/2 \times 7.567 \times 1.00 \times 1.20 \times 2.4) \times 0.1$	$= 2.47^t$
Beam	$1.00^2 \times 0.30 \times 2.4 \times 0.1$	$= 0.07$ "
Haunch	$1/2 \times 1.50 \times 0.30 \times 1.00 \times 2.4 \times 2 \times 0.1$	$= 0.11$ "
Haunch	$1/2 \times 0.60^2 \times 1.20 \times 2.4 \times 0.1$	$= 0.05$ "
		$P_{H1} = 2.70^t$

Pile $\gamma_{H1} = \pi/4 \times 0.914^2 \times 2.40^{t/m^3} \times 0.1$ $= 0.16^{t/m}$

o Dead Weight of Pile

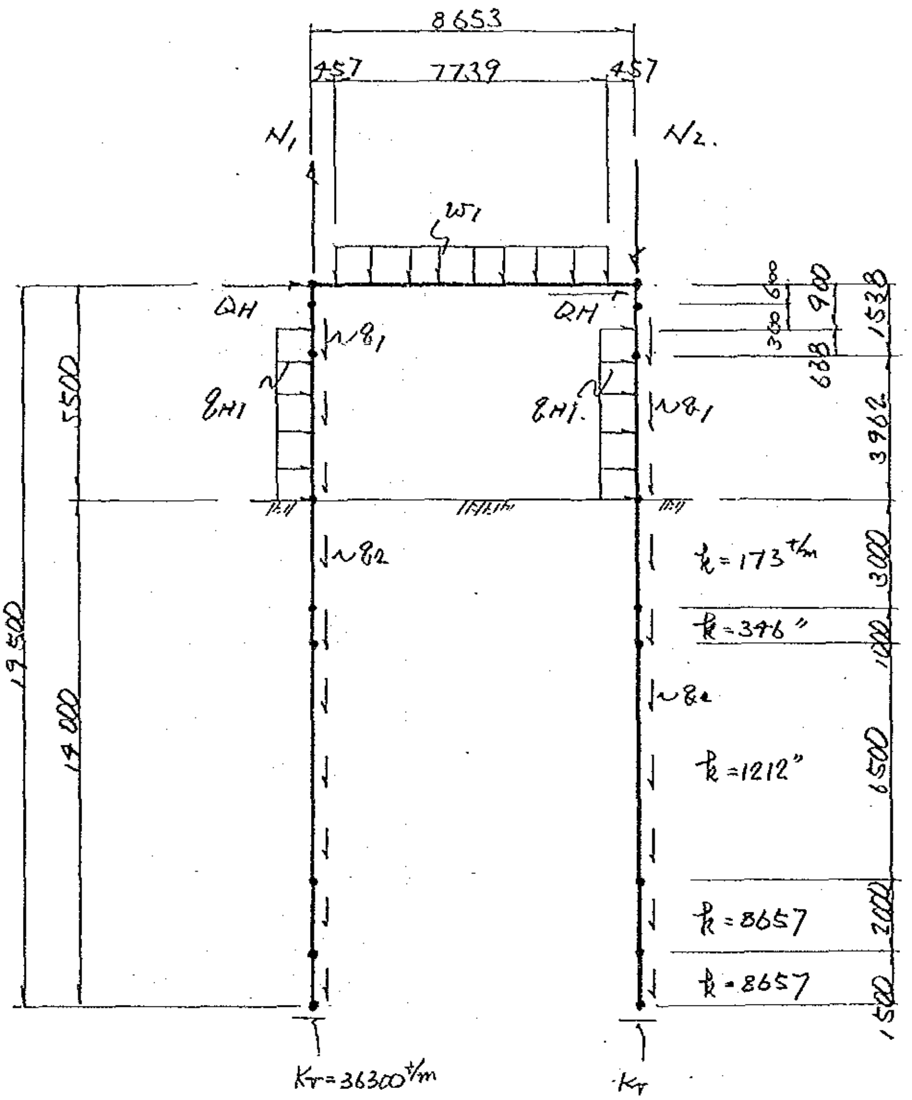
In the air $\gamma_1 = \pi/4 \times 0.914^2 \times 2.4^{t/m^3}$ $= 1.57^{t/m}$

In the water $\gamma_2 =$ " 1.4 " $= 0.92$ "

o Lateral Force on Tower $(k_H = 0.1)$

$P_{H2} = 26.54 \times 0.1$ $= 2.65^t$

Load Diagram (during earthquakes C-T) CASE 4



$N_1 = P_1 - T = 16.10 - 22.82$

$= -6.72^t$

$N_2 = P_1 + C = 16.10 + 19.29$

$= 35.39^t$

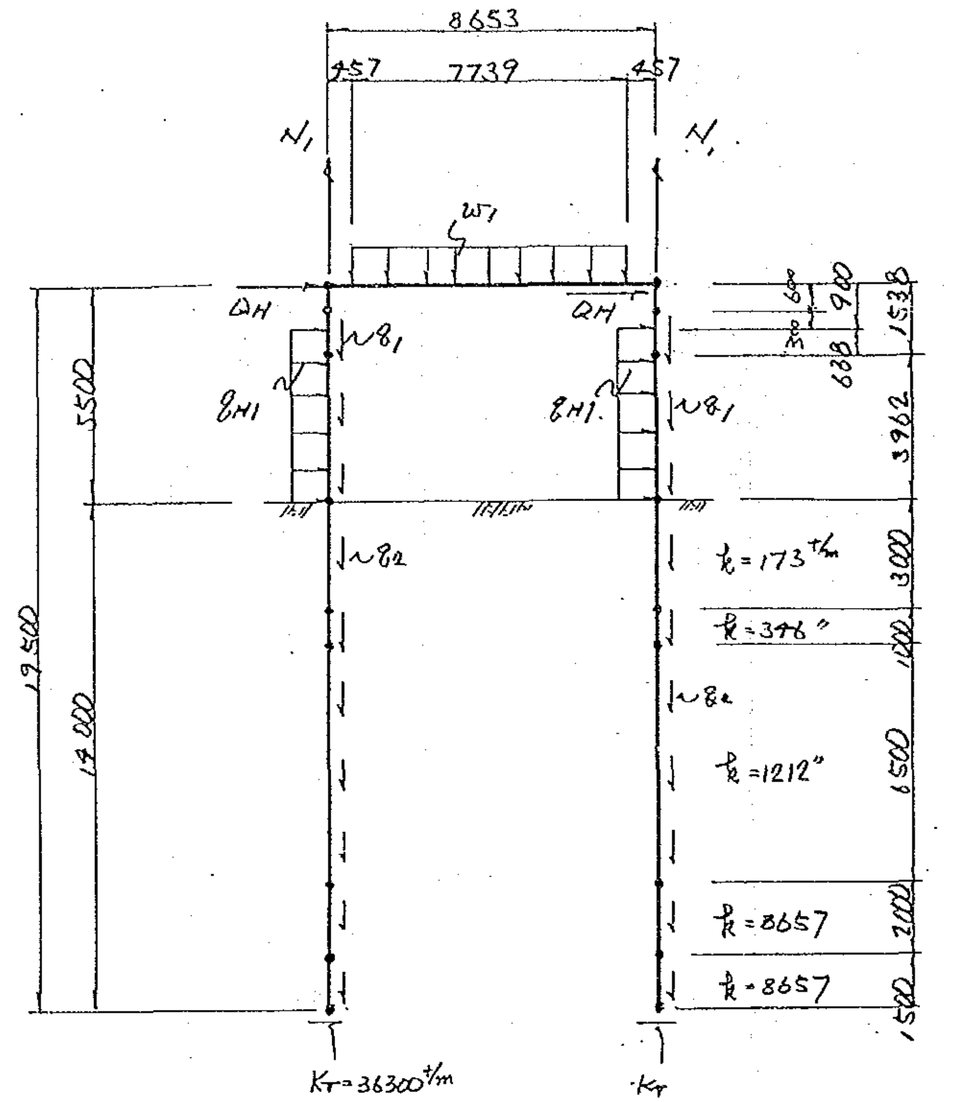
$Q_H = Q + P_{H1} + P_{H2} = 2.05 + 2.70 + 2.65$

$= 7.40^t$

15

No. ....

Load Diagram (during earthquakes T-T) CASE 5



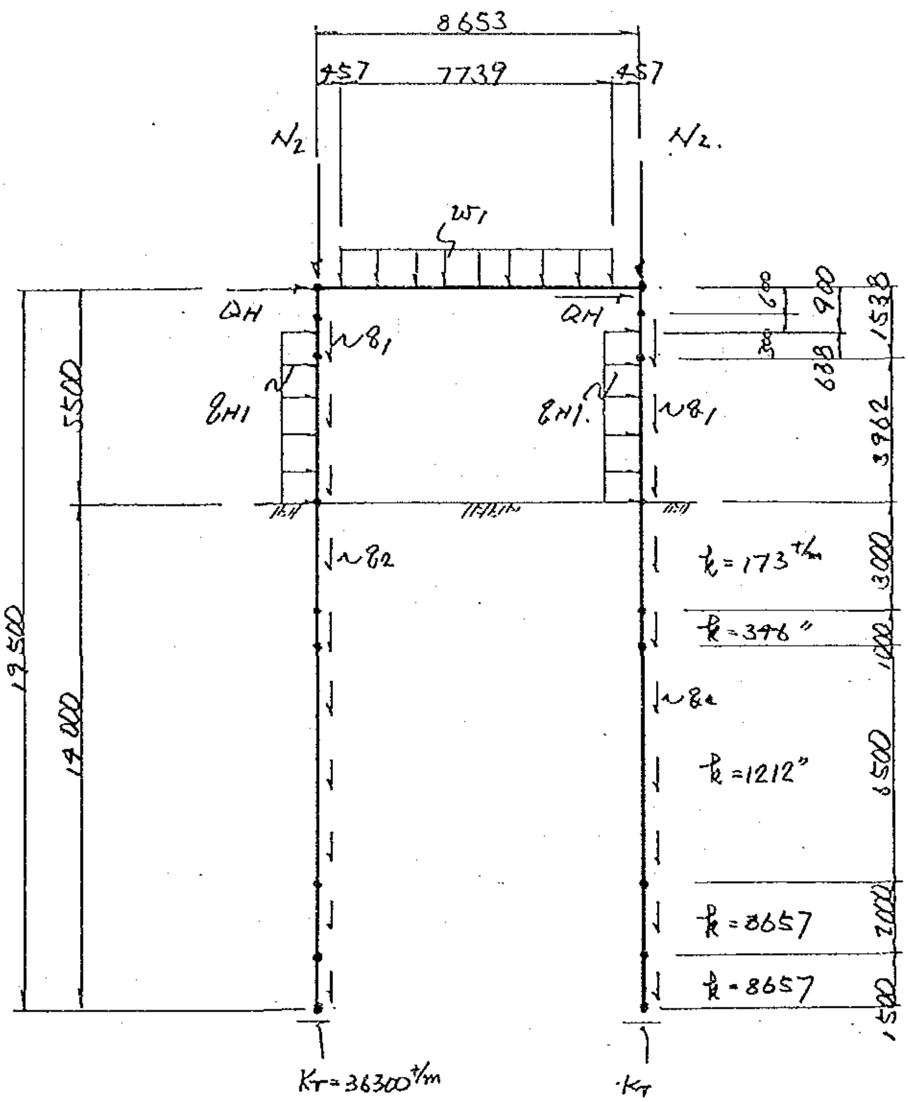
$N_1 = -6.72^t$

$Q_H = Q + P_{H1} + P_{H2} = 2.05 + 3.98 + 2.70$

$= 7.40^t$

177

Load Diagram (during earthquakes C-C) CASE 6



$N_2 = 35.39^t$

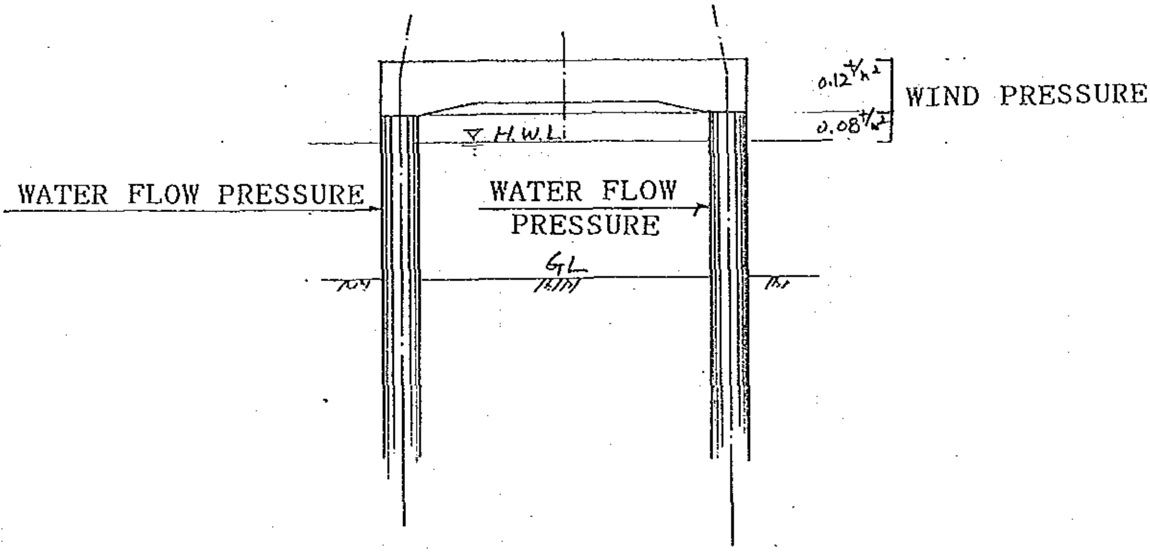
$Q_H = Q + P_{H1} = 2.05 + 2.70 + 2.65$

$= 7.40^t$

17

(i) During Floods

Load Computation



o Design Load

$C = 55.06^t$

$T = 37.28^t$

$Q = 6.783 + 4.489 = 11.28^t$

o Dead Weight of Foundation

$w_1 = 2.88^{t/m}$

$p_1 = 14.50^t$

$p_2 = 1.08$ "

$p_3 = 0.52$ "

$P_1 = 16.10^t$

174

o Wind Pressure

$$P_{H1} = \{1/2 \times 9.567 \times 120 + 1/2 \times (1.00 + 2.50) \times 0.30\} \times 0.12 \ t/m^2$$

$$= 0.75 \ t$$

$$q_{H1} = 0.914 \times 0.08 \ t/m^2 \qquad = 0.07 \ t/m$$

o Water Flow Pressure

The water flow pressure is calculated using the following formula.

$$P_{H2} = K \cdot A \cdot V^2 \quad (+)$$

where,

K : coefficient of sectional shape (circular section = 0.04)

A : vertical projected area of pile

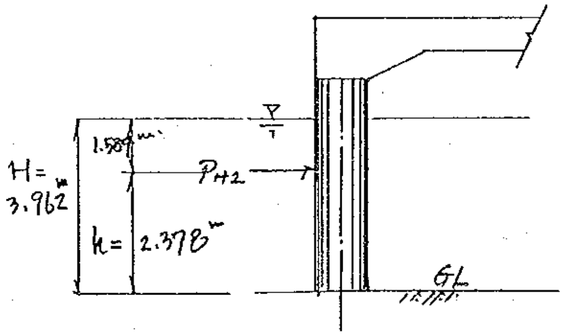
V : flow velocity (v = 1.83 m/s)

H : depth of water (= 3.962 m)

The application point of the water flow pressure is 0.6H above ground level.

$$\therefore P_{H2} = 0.04 \times 0.914 \times 1.83^2 \times 3.962 \qquad = 0.49^{t}$$

Application point (h) = 3.962 × 0.6 = 2.378 m



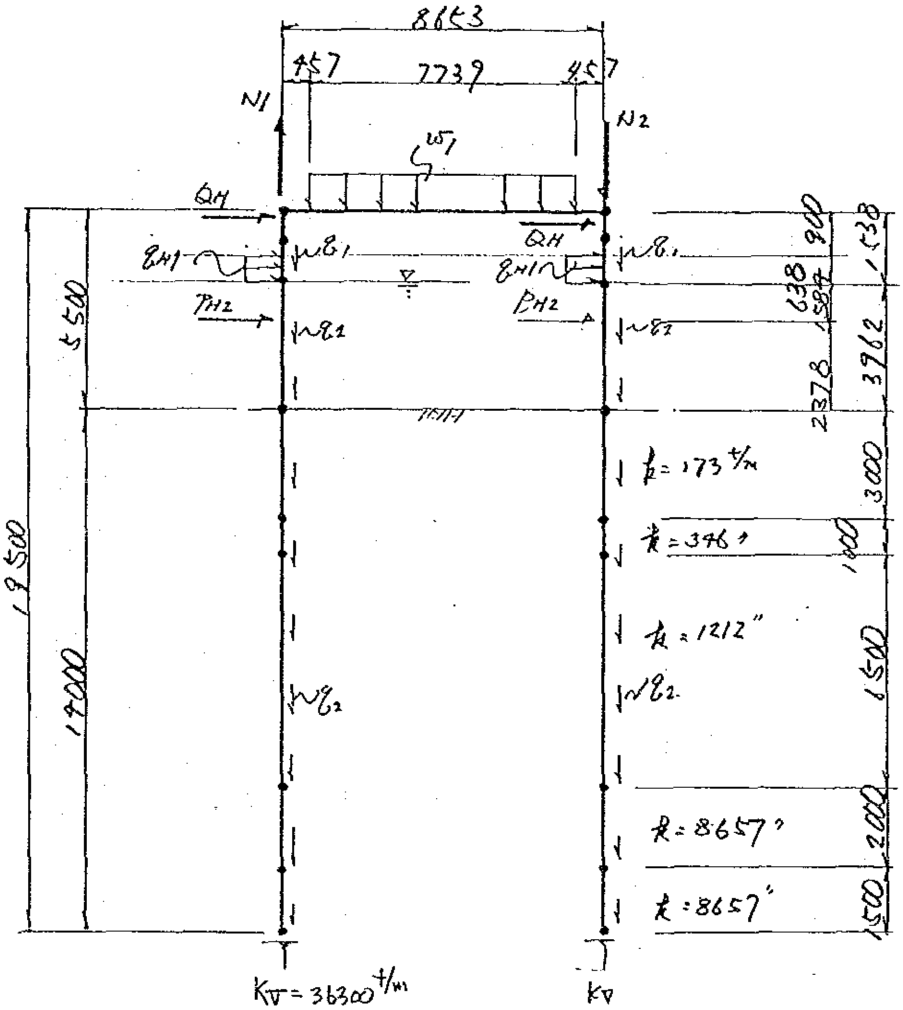
No.

o Dead Weight of Pile

In air $\gamma_1 = 1.57$ t/m

In water $\gamma_2 = 0.92$ 〃

Load Diagram (during floods C-T) CASE 7



$N1 = P_1 - T = 16.10 - 55.06 = -38.96^t$

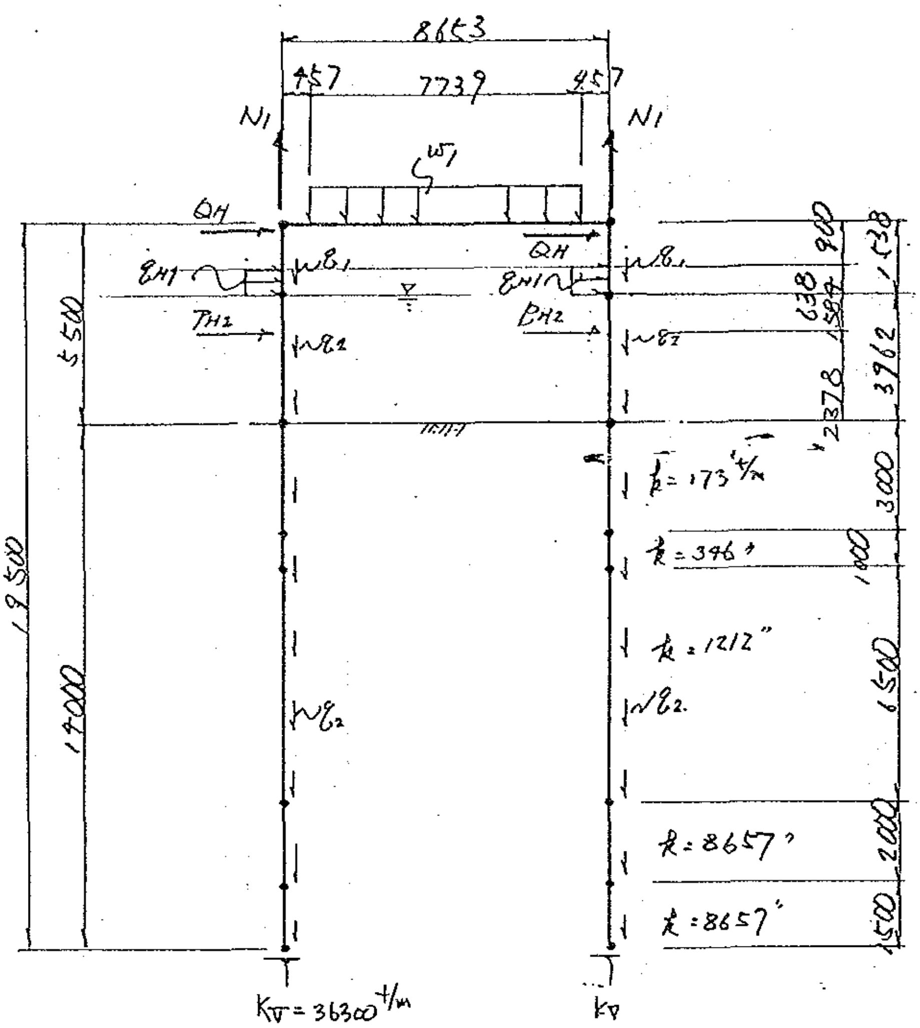
$N2 = P_1 + C = 16.10 + 37.28 = 53.38^t$

$Q_H = Q + P_{H1} = 11.28 + 0.75 = 12.03^t$

$P_{H2} = 0.49^t$

21

Load Diagram (during floods T-T) CASE 8



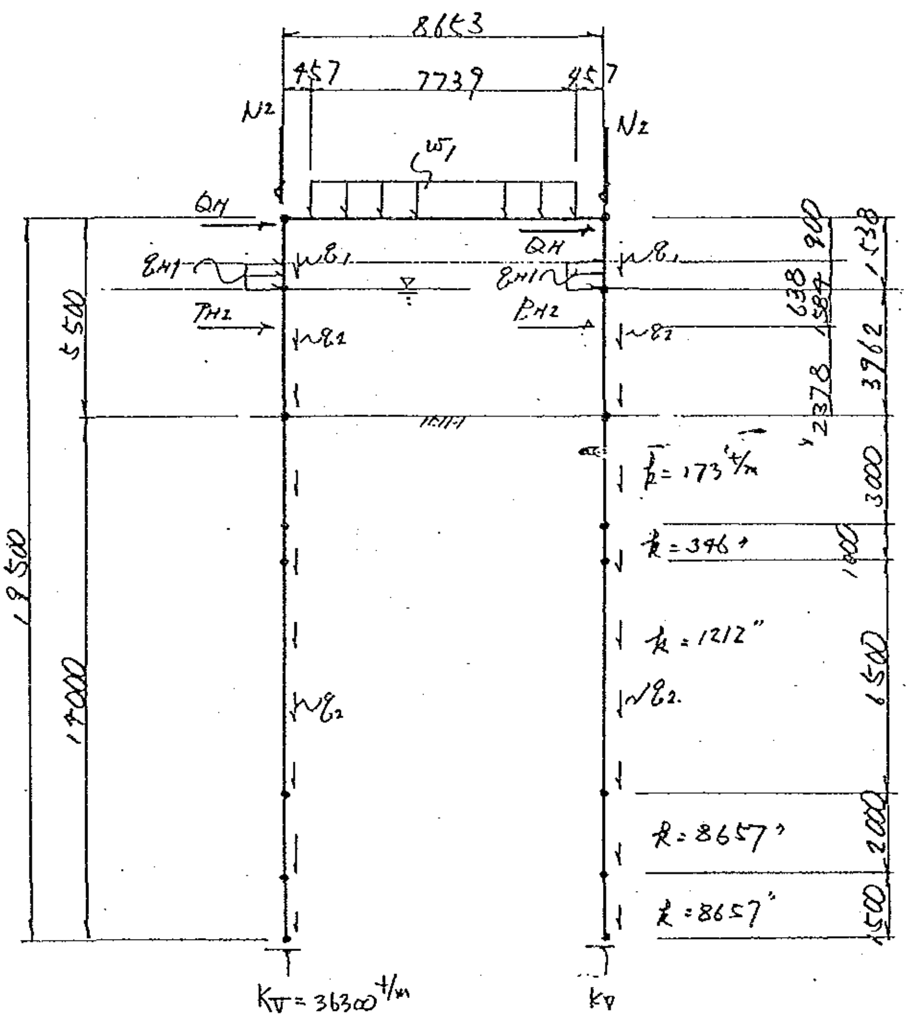
$N_1 = -38.96^t$

$Q_H = Q + P_{H1} = 11.28 + 0.75 = 12.03^t$

$P_{H2} = 0.49^t$

178

Load Diagram (during floods C-C) CASE 9



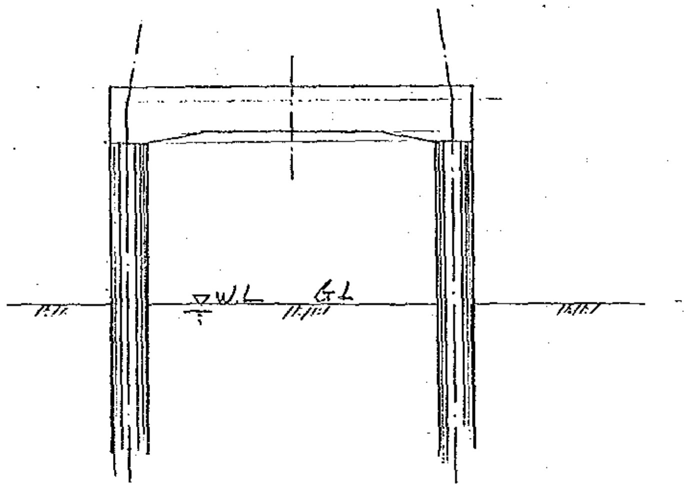
$N_2 = 53.38^t$

$Q_H = Q + P_{H1} = 11.28 + 0.75 = 12.03^t$

$P_{H2} = 0.49^t$

(j) During Load E

Load Computation



o Design Loads

$C = 19.29^{t}$

$T = 22.82^{t}$

$Q = 1.341 + 0.704 = 2.05^{t}$

o Dead Weight of Foundation

$w_1 = 2.88\ ^{t}/_{m}$

$P_1 = 14.50^{t}$ (Beam)

$P_2 = 1.08$ " (Haunch)

$P_3 = 0.52$ " (Horizontal haunch)

$P = 16.10^{t}$

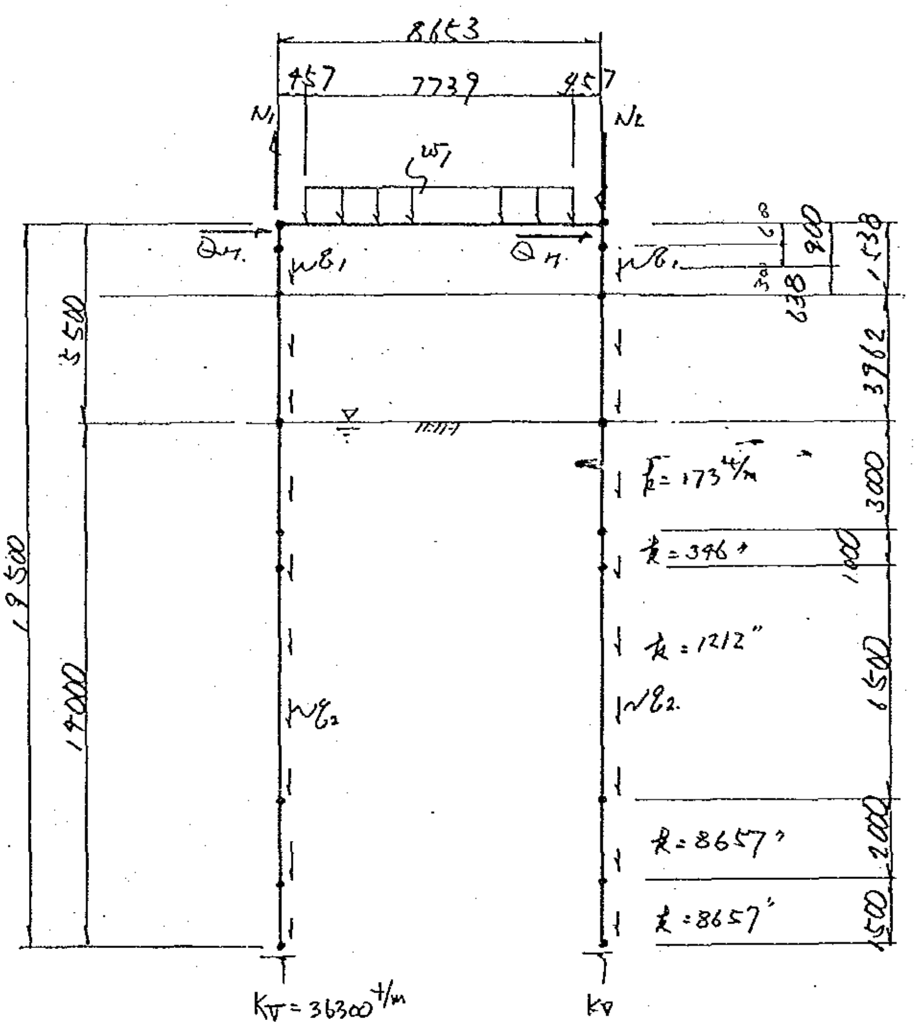
o Pile Weight

In air $\gamma_1 = 1.57\ ^{t}/_{m}$

In water $\gamma_2 = 0.92$ "

No. ....

Load Diagram (during load E C-T) CASE 10



$N_1 = P_1 - T = 16.10 - 22.82 = -6.72^t$

$N_2 = P_1 + C = 16.10 + 19.29 = 35.39^t$

$Q_H = Q = 2.05^t$

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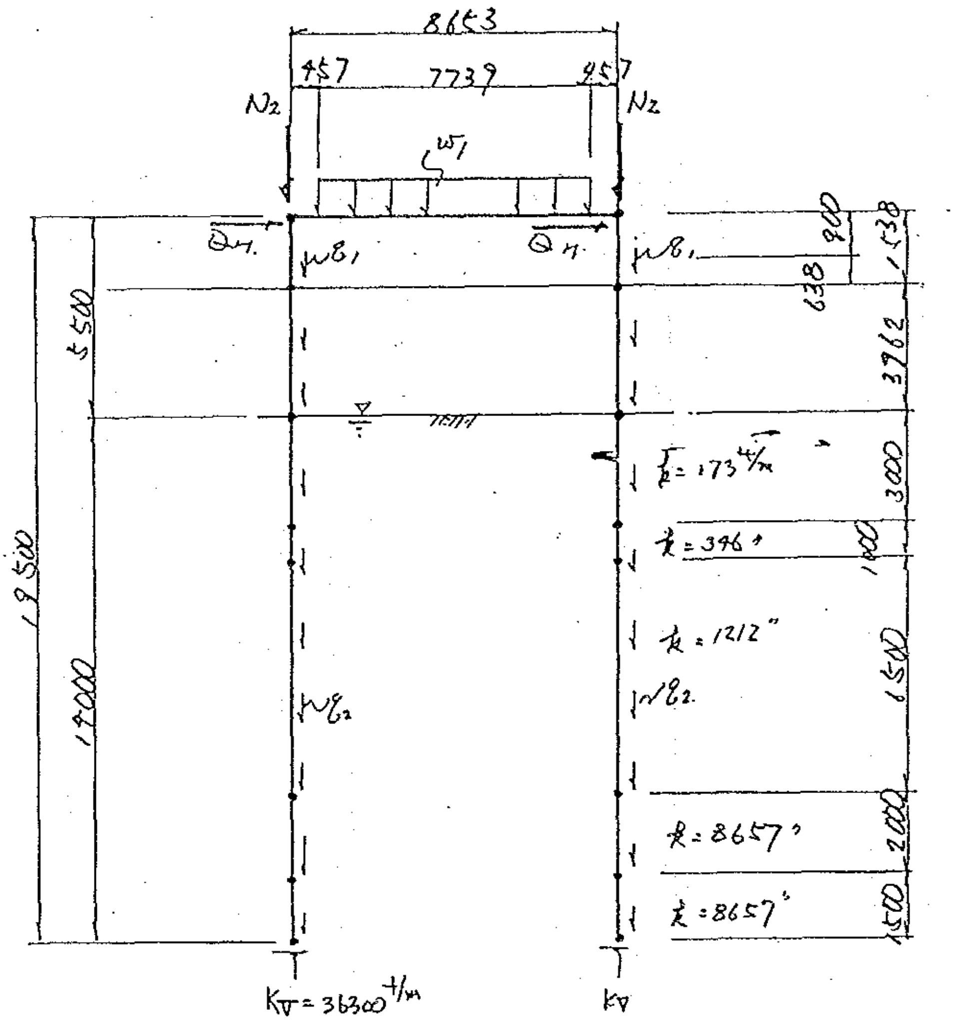
Load Diagram (during load E C-C) CASE 11



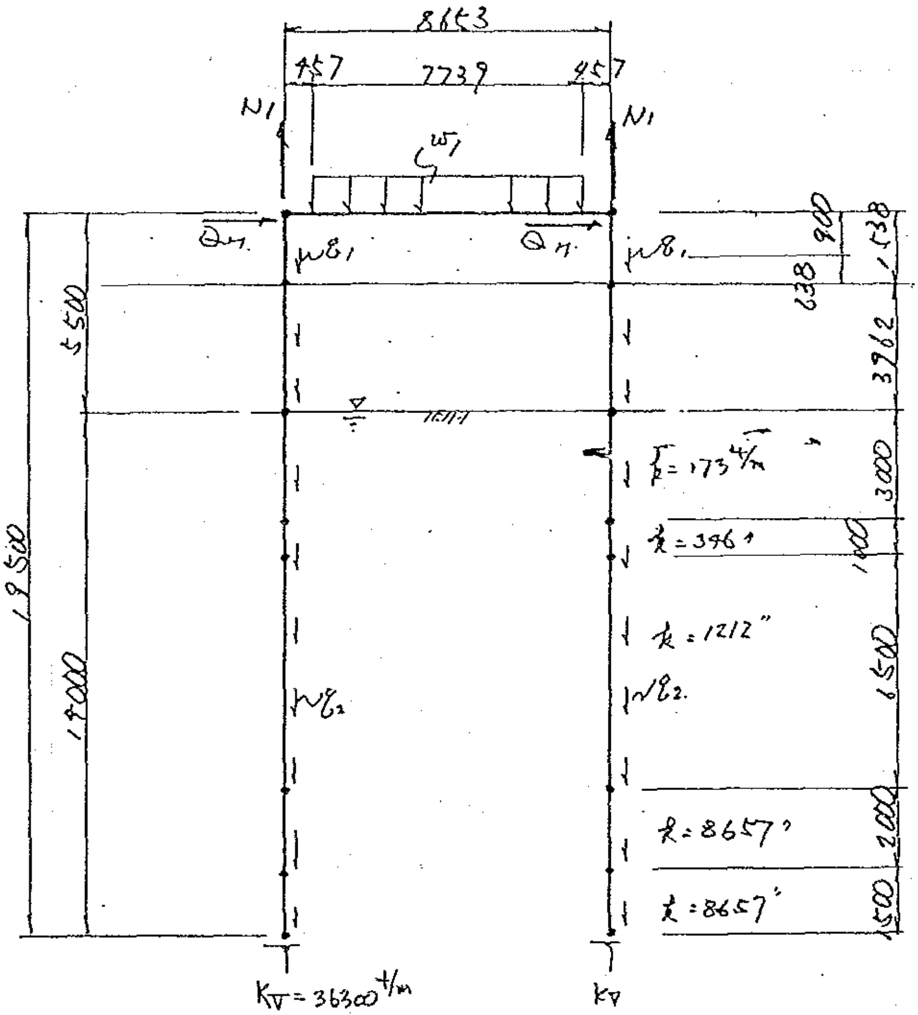
$N_2 = 35.39^t$

$Q_H = Q = 2.05^t$

182

No. ....

Load Diagram (during load E T-T) CASE 12



$N_1 = -6.72^t$

$Q_H = Q = 2.05^t$

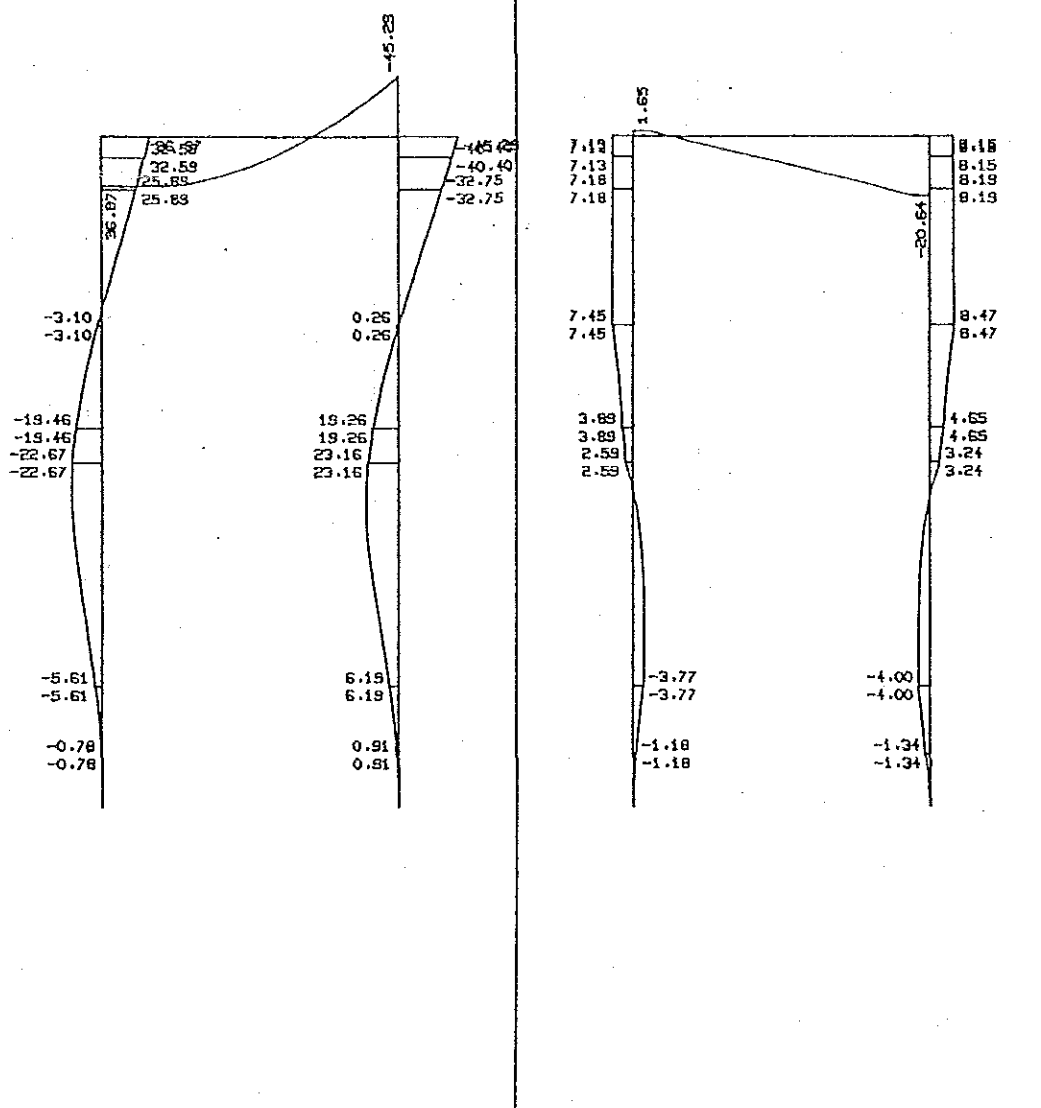
27

(k) OUT PUT

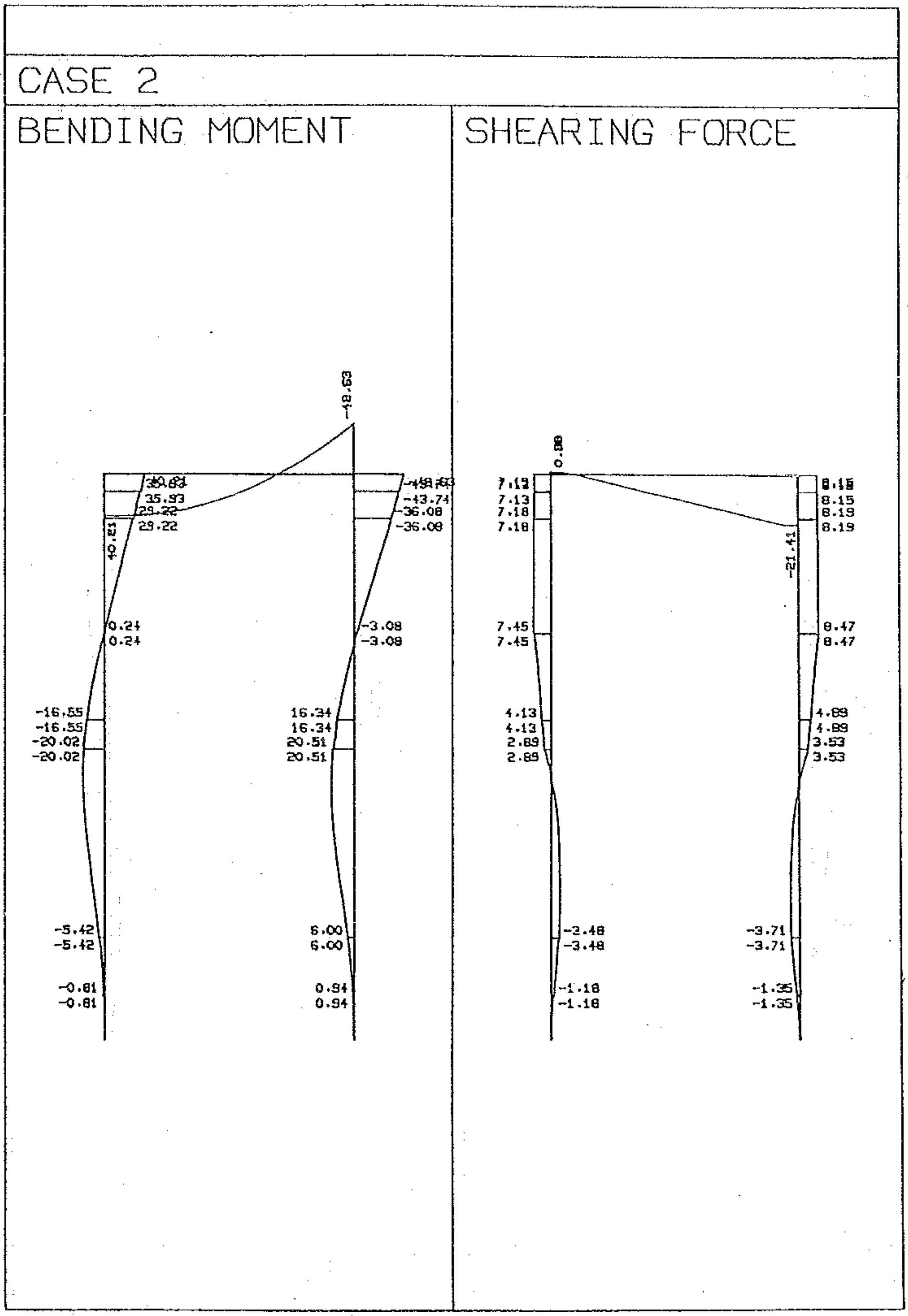
CASE 1

BENDING MOMENT

SHEARING FORCE



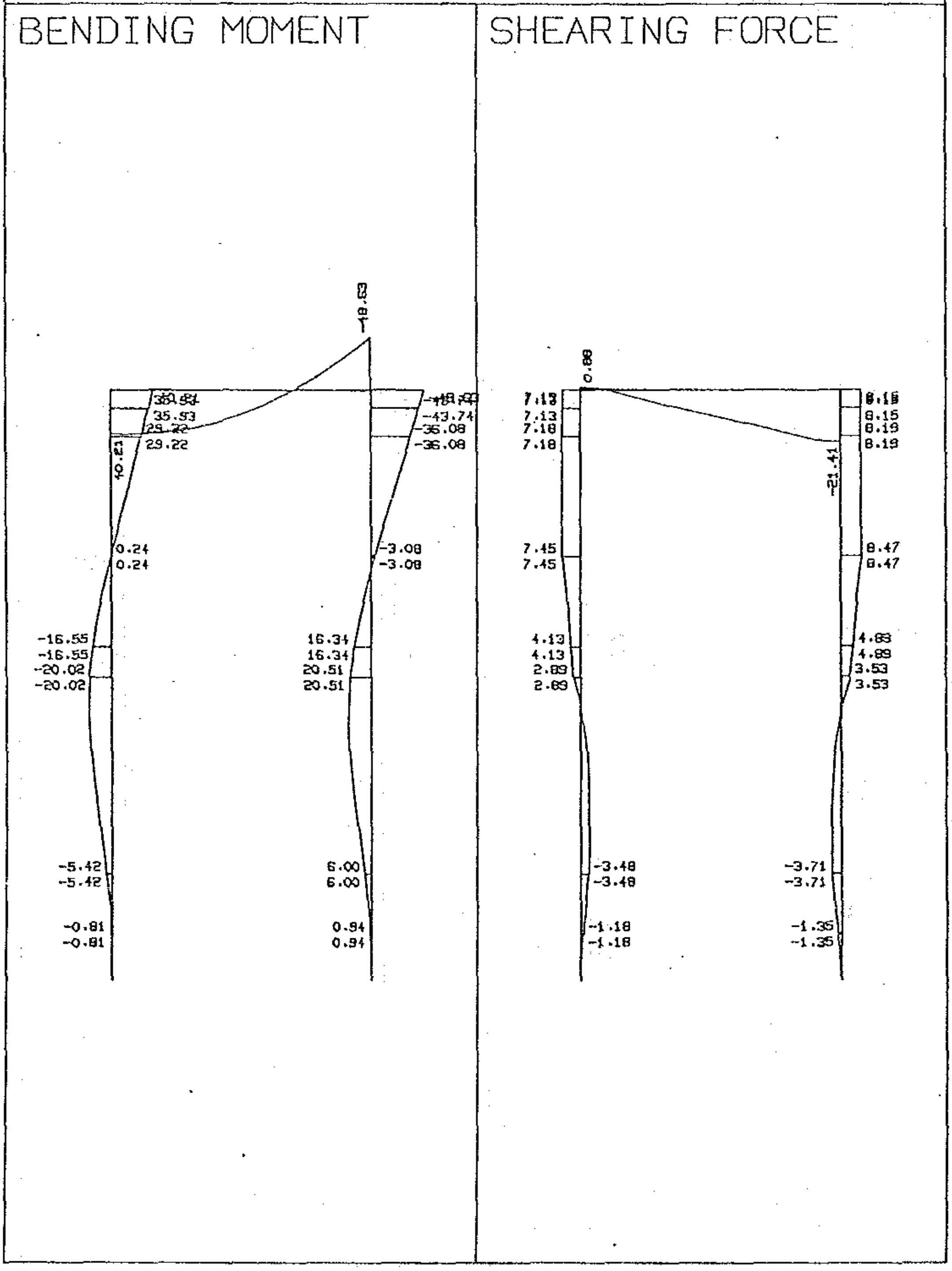
184


CASE 2
BENDING MOMENT
SHEARING FORCE
-48.63
35.93
29.22
40.81
-43.74
-36.08
0.24
-3.08
-16.55
-20.02
16.34
20.51
-5.42
6.00
-0.81
0.94
0.88
7.13
7.18
8.15
8.19
-21.41
7.45
8.47
4.13
2.89
4.89
3.53
-3.48
-3.71
-1.18
-1.35

29

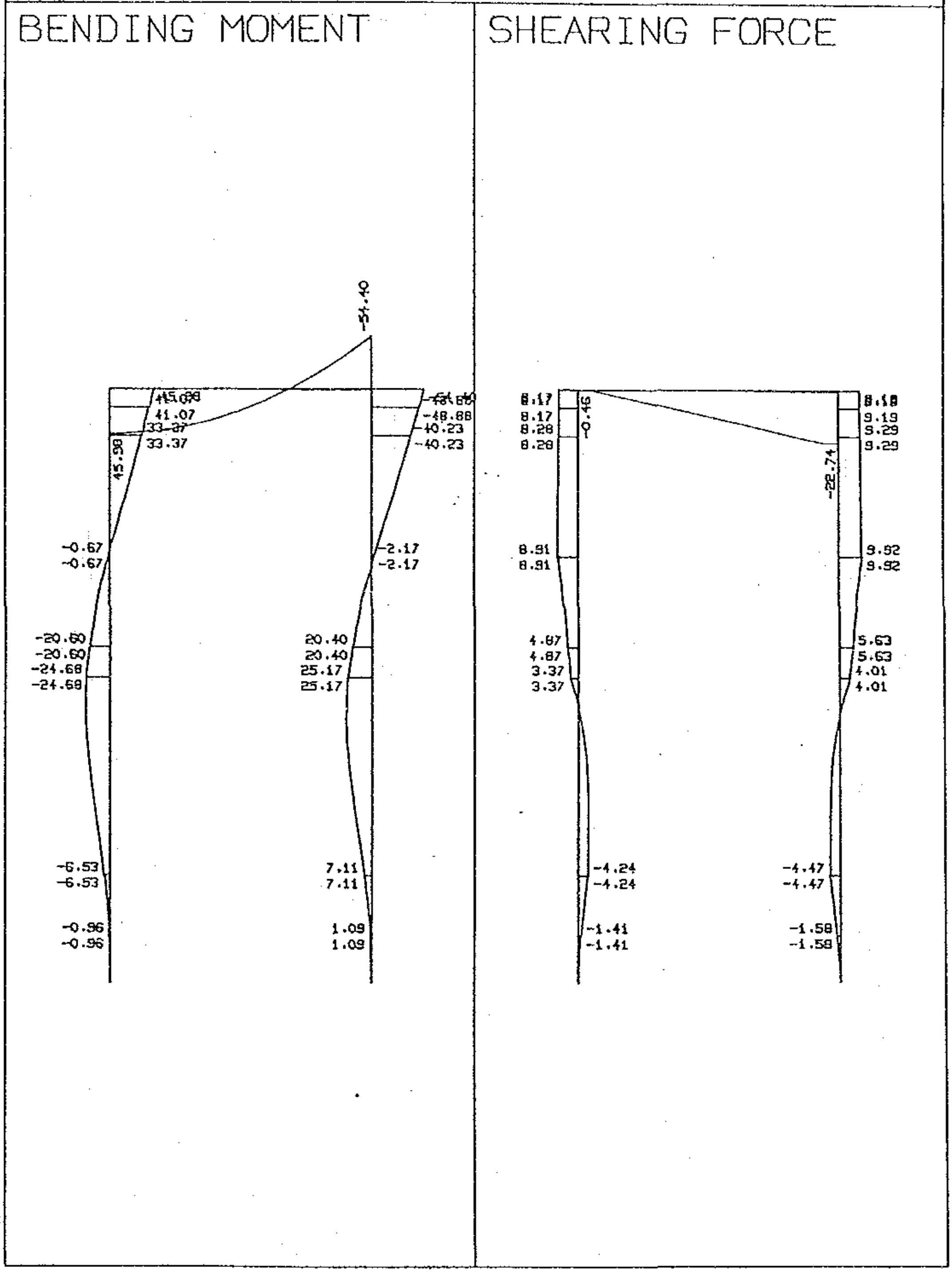
58/

CASE 3



196

CASE 4


BENDING MOMENT
-54.40
41.07
33.37
33.37
45.98
-48.88
-40.23
-40.23
-0.67
-0.67
-2.17
-2.17
-20.60
-20.60
-24.68
-24.68
20.40
20.40
25.17
25.17
-6.53
-6.53
7.11
7.11
-0.96
-0.96
1.09
1.09
SHEARING FORCE
8.17
8.17
8.28
8.28
-0.46
9.19
9.29
9.29
-22.74
8.91
8.91
9.92
9.92
4.87
4.87
3.37
3.37
5.63
5.63
4.01
4.01
-4.24
-4.24
-4.47
-4.47
-1.41
-1.41
-1.58
-1.58

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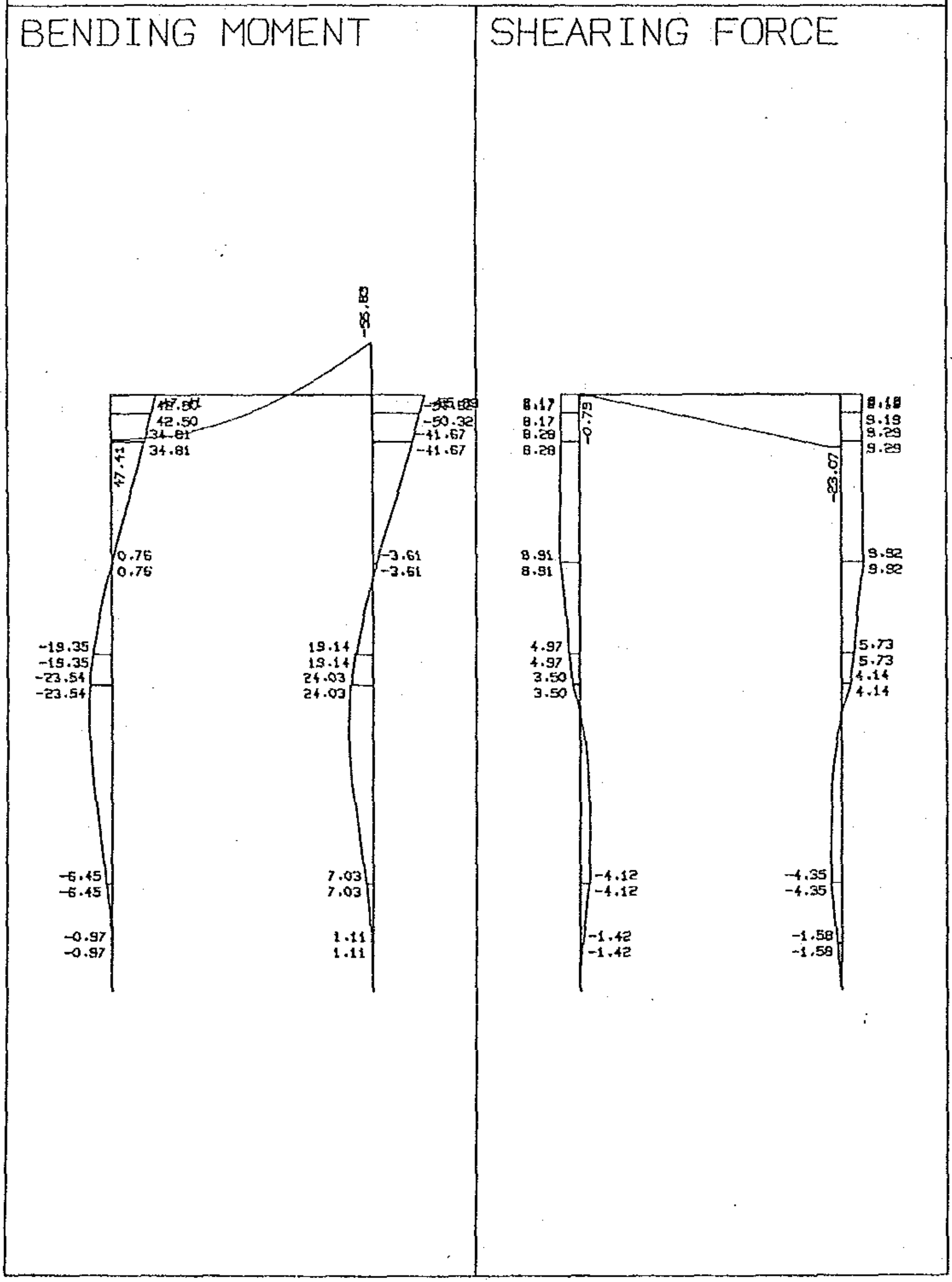
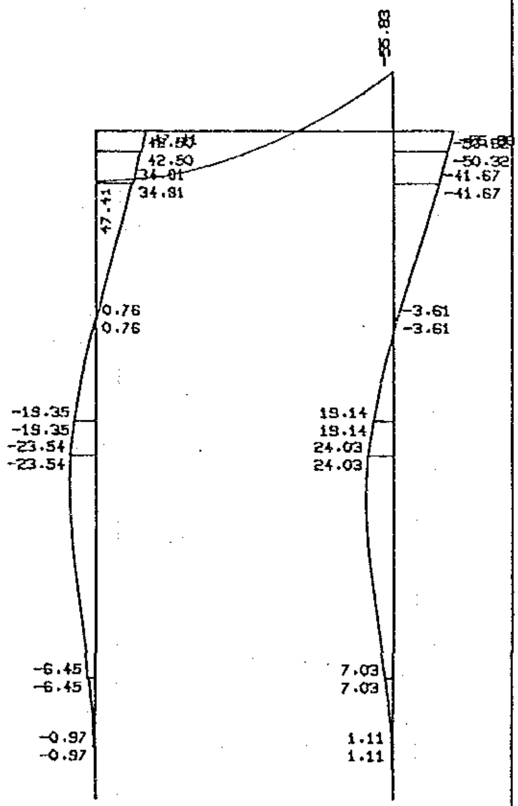
CASE 5

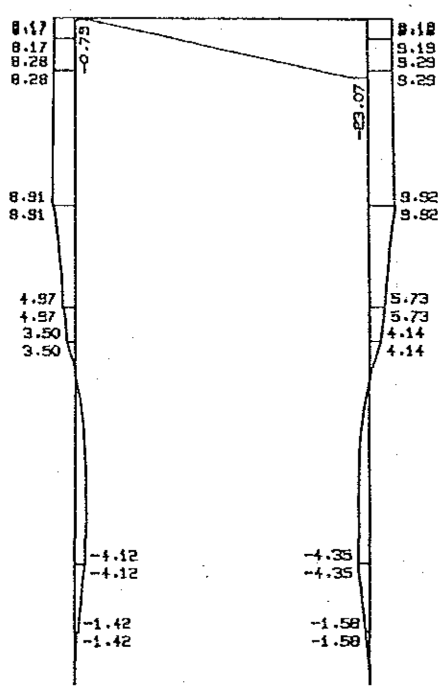

BENDING MOMENT
SHEARING FORCE
-95.83
42.50
34.81
34.81
47.41
-50.32
-41.67
-41.67
0.76
0.76
-3.61
-3.61
-19.35
-19.35
-23.54
-23.54
19.14
19.14
24.03
24.03
-6.45
-6.45
7.03
7.03
-0.97
-0.97
1.11
1.11
8.17
8.17
8.28
8.28
-0.79
9.19
9.29
9.29
-23.67
8.91
8.91
9.92
9.92
4.97
4.97
3.50
3.50
5.73
5.73
4.14
4.14
-4.12
-4.12
-4.35
-4.35
-1.42
-1.42
-1.58
-1.58

CASE 6

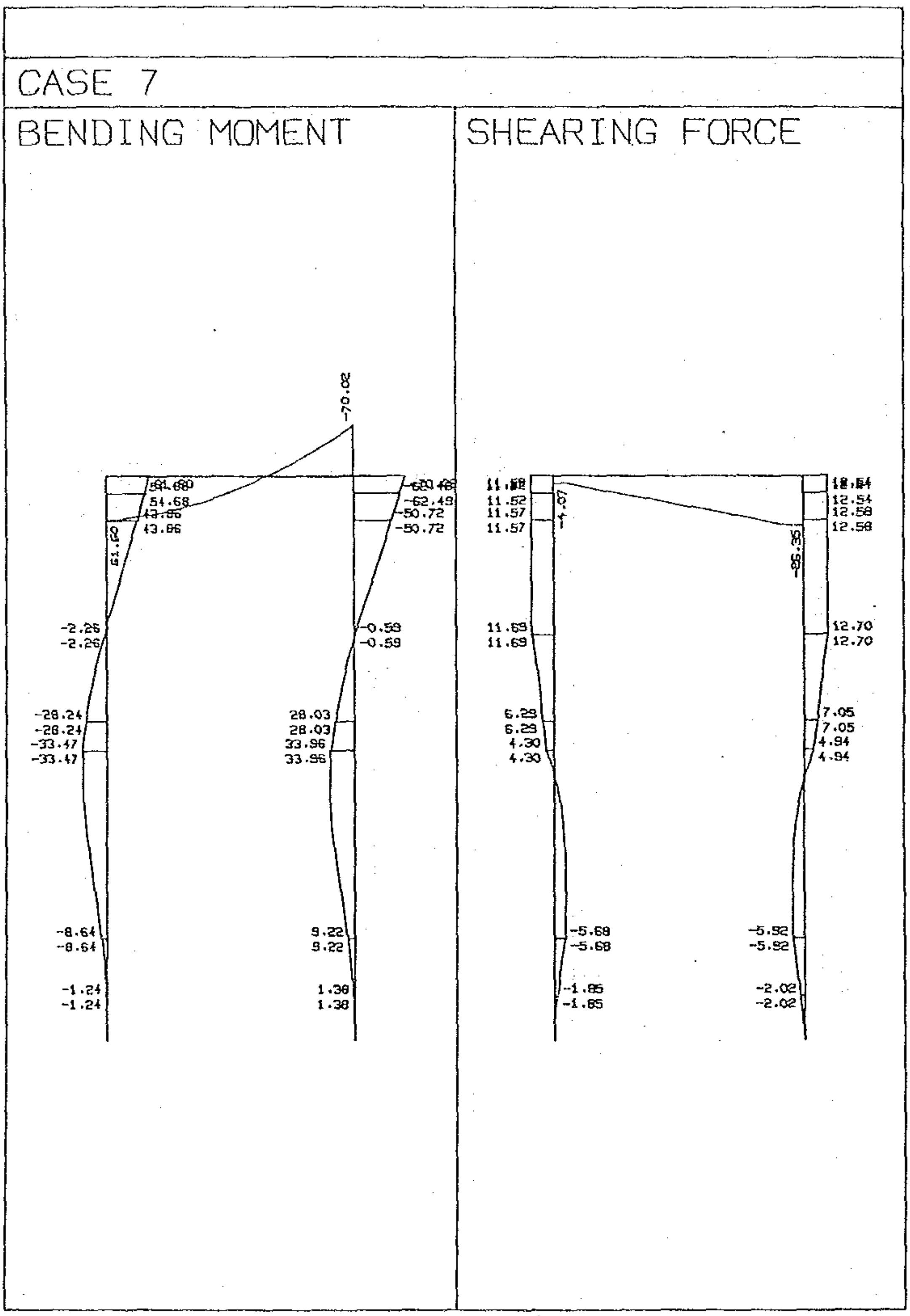
BENDING MOMENT



SHEARING FORCE



/8/


CASE 7
BENDING MOMENT
SHEARING FORCE
-70.02
54.68
43.86
43.86
61.60
-62.49
-50.72
-50.72
-2.26
-2.26
-0.59
-0.59
-28.24
-28.24
-33.47
-33.47
28.03
28.03
33.96
33.96
-8.64
-8.64
9.22
9.22
-1.24
-1.24
1.38
1.38
11.52
11.57
11.57
-4.07
12.54
12.58
12.58
-85.35
11.69
11.69
12.70
12.70
6.29
6.29
4.30
4.30
7.05
7.05
4.94
4.94
-5.68
-5.68
-5.92
-5.92
-1.85
-1.85
-2.02
-2.02

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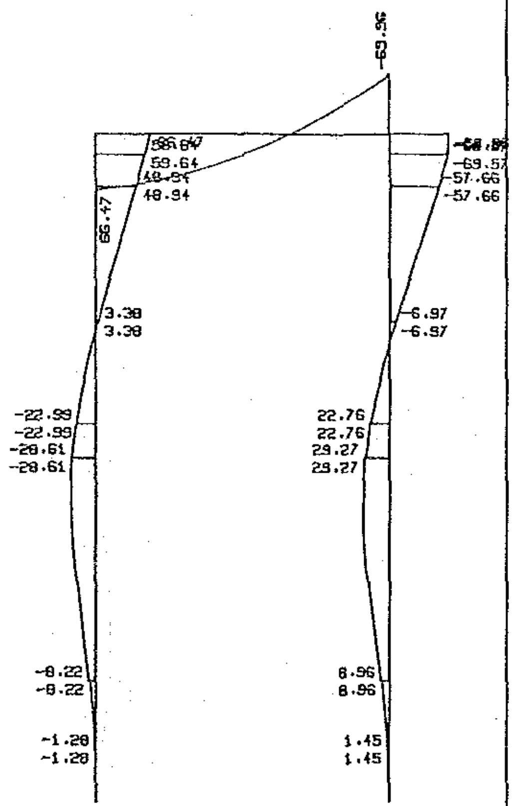
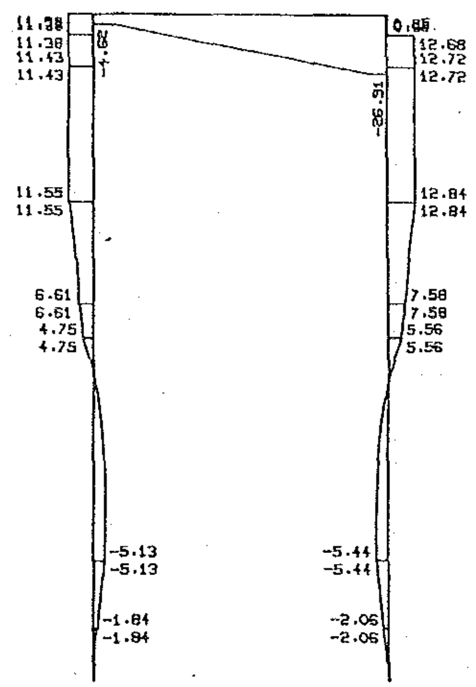
CASE 8

BENDING MOMENT



SHEARING FORCE



35

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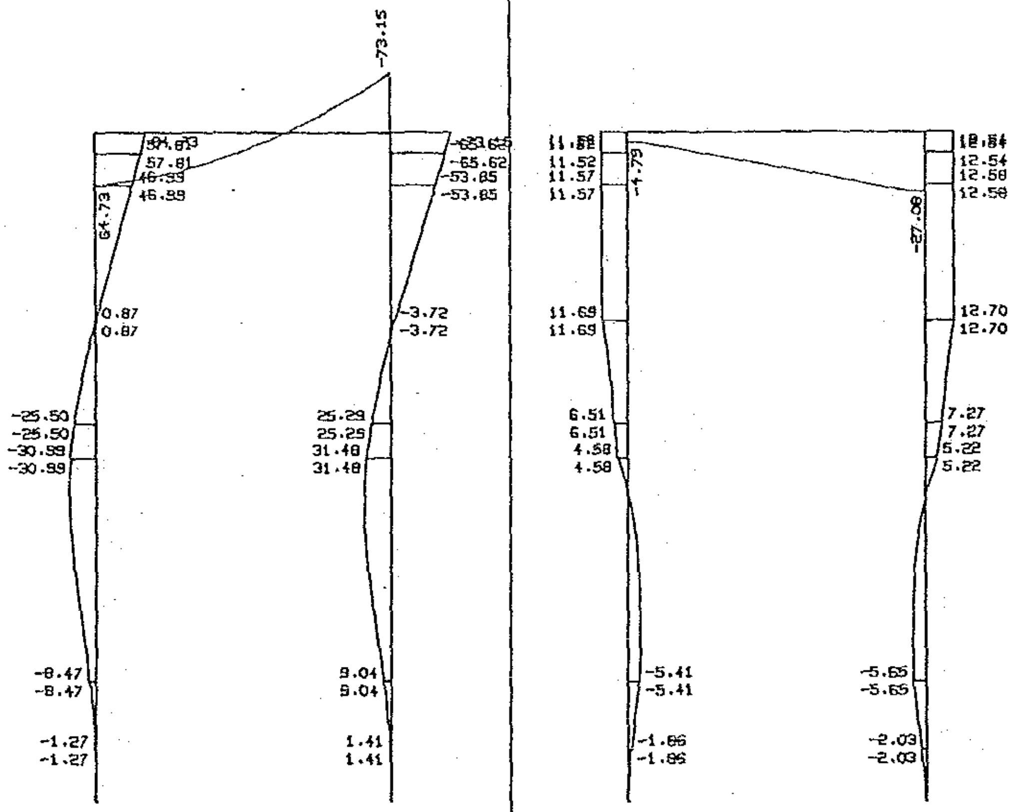
CASE 9

BENDING MOMENT

SHEARING FORCE



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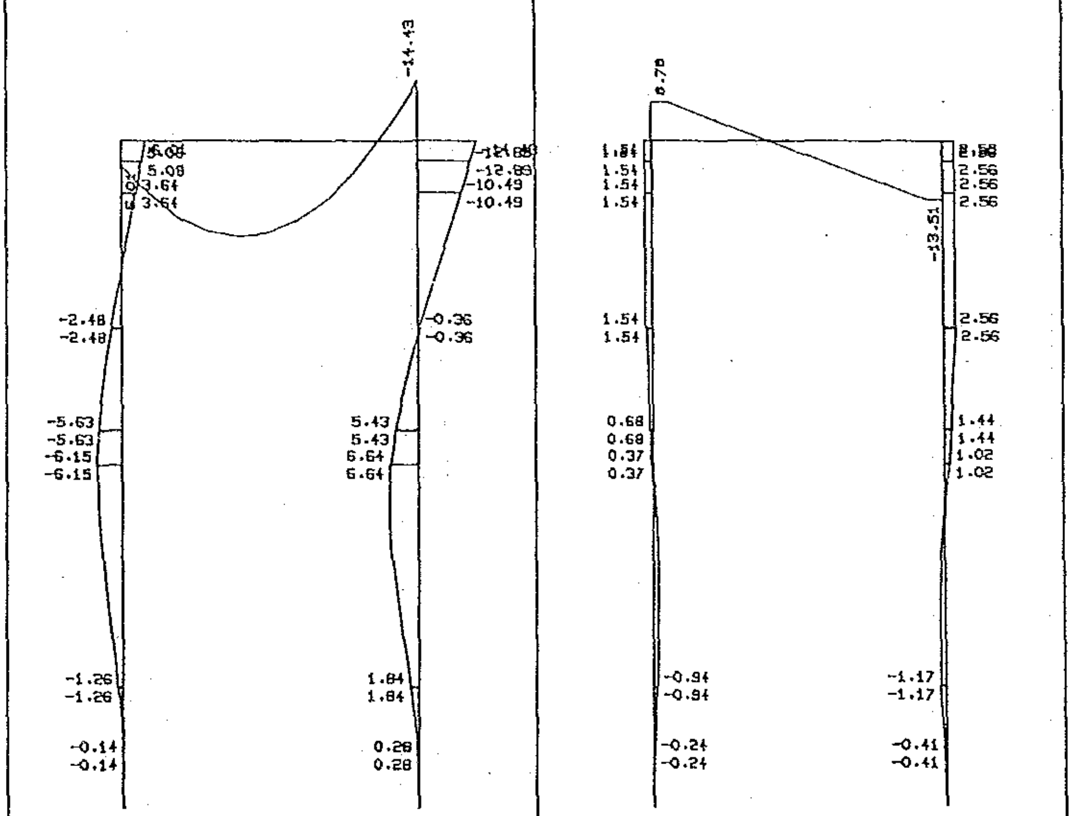
CASE 1Ø

BENDING MOMENT | SHEARING FORCE



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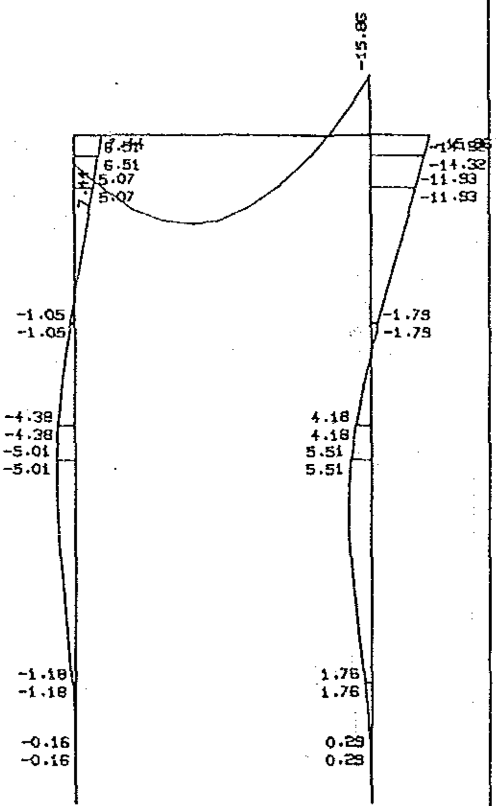
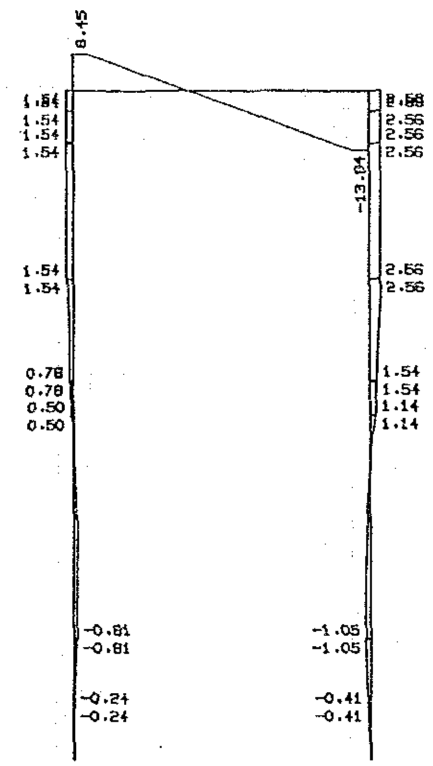
CASE 11



BENDING MOMENT
-15.86
6.51
5.07
5.07
-14.32
-11.93
-11.93
-1.05
-1.05
-1.79
-1.79
-4.38
-4.38
-5.01
-5.01
4.18
4.18
5.51
5.51
-1.18
-1.18
1.76
1.76
-0.16
-0.16
0.29
0.29



SHEARING FORCE
8.45
1.54
1.54
1.54
2.56
2.56
2.56
-13.04
1.54
1.54
2.56
2.56
0.78
0.78
0.50
0.50
1.54
1.54
1.14
1.14
-0.81
-0.81
-1.05
-1.05
-0.24
-0.24
-0.41
-0.41

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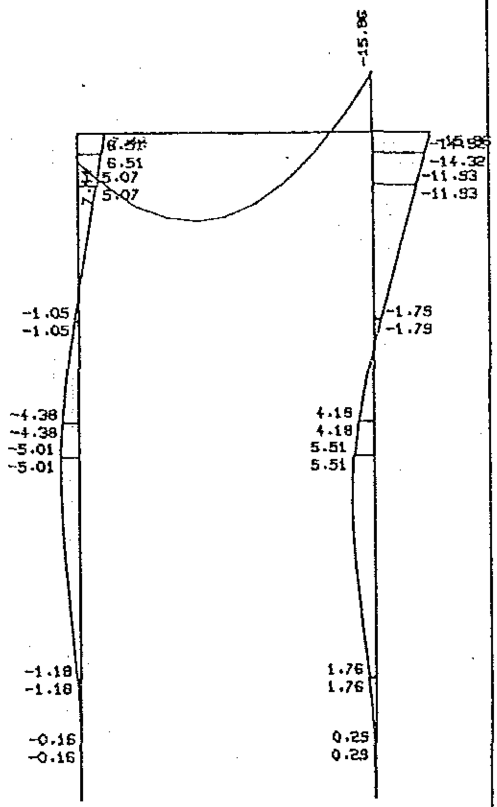
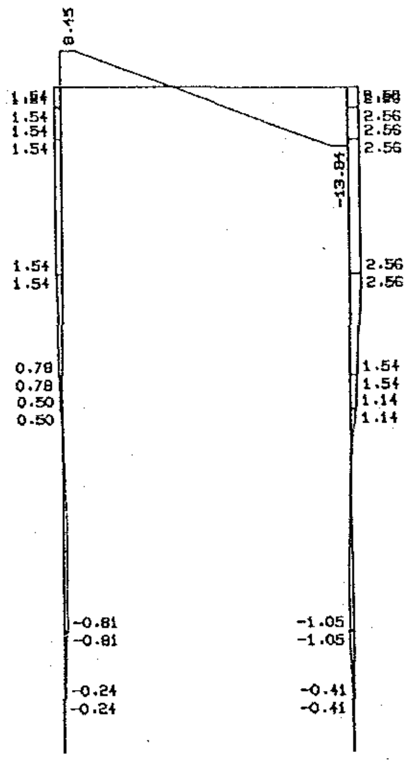
CASE 12

BENDING MOMENT



SHEARING FORCE



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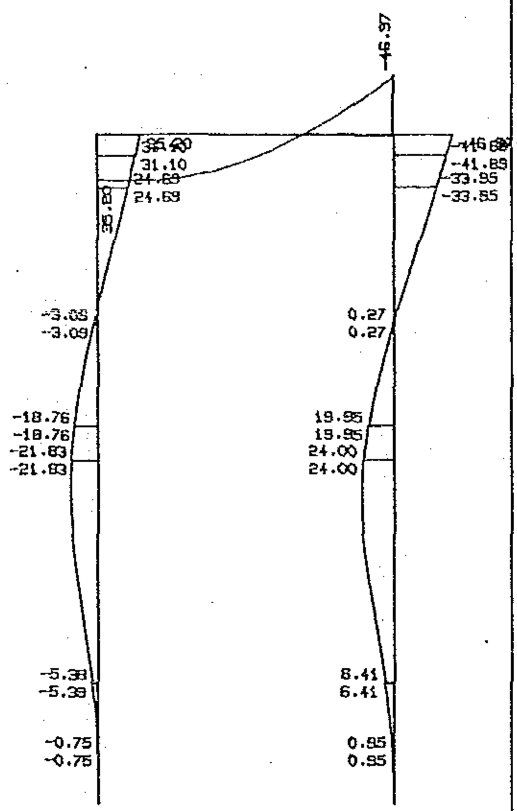
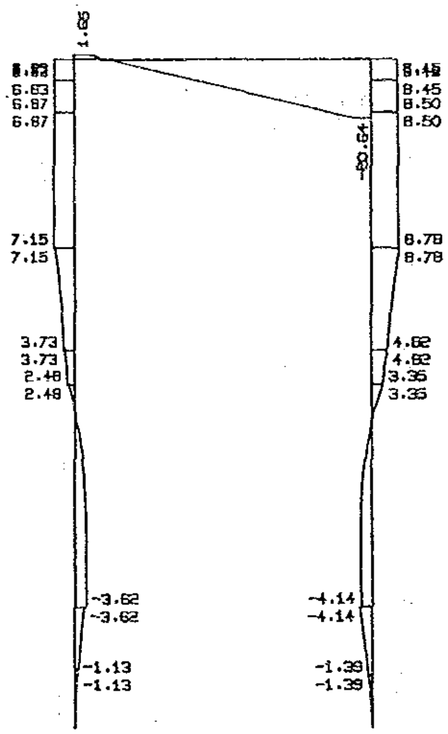
CASE 13 (TEMPERATURE CHANGES)

BENDING MOMENT



SHEARING FORCE



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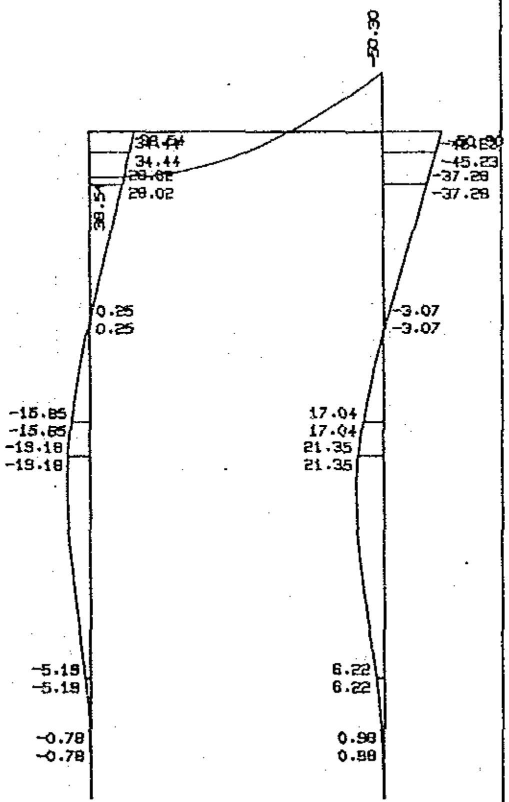
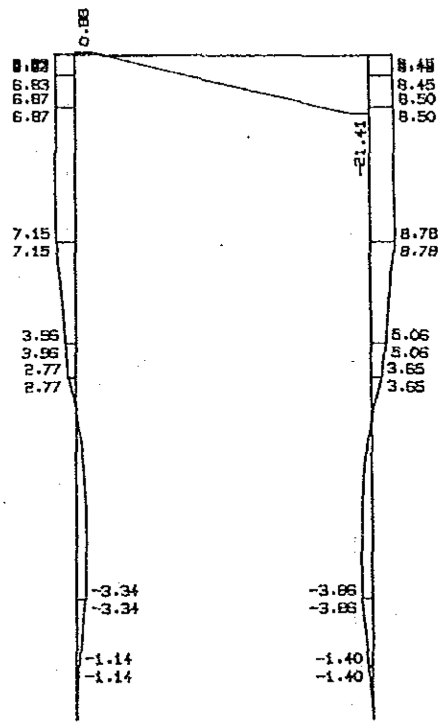
CASE 14 (TEMPERATURE CHANGES)

BENDING MOMENT



SHEARING FORCE



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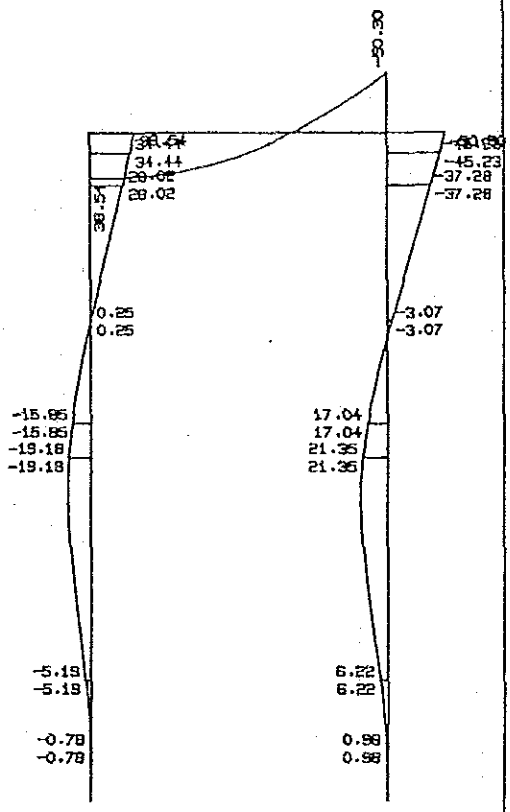
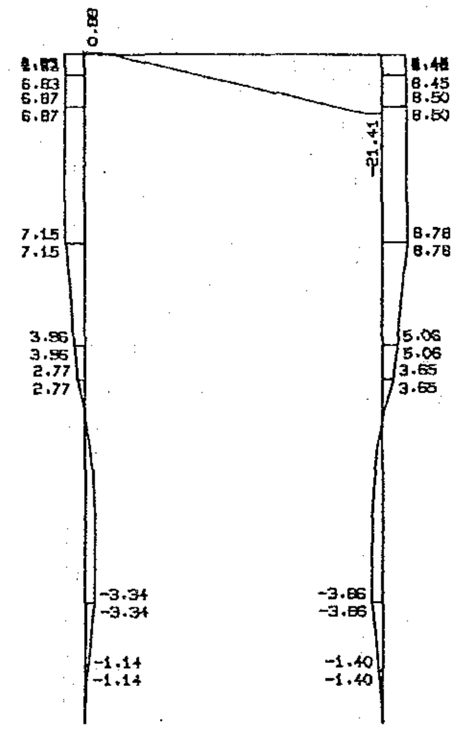
CASE 15 (TEMPERATURE CHANGES)

BENDING MOMENT



SHEARING FORCE



(1) Pile Reaction

Types / Cases	C — T		T — T		C — C		Displacement (mm)	
	N_{max} (+)	N_{min} (+)	N_{max} (+)	N_{min} (+)	N_{max} (+)	N_{min} (+)	Top	Top of G.L.
Oblique Wind	116.20	13.79	18.86	14.56	116.98	—	19.7	10.9
Oblique Wind	116.20	13.79	18.86	14.56	116.98	—	20.4	11.2
Earthquakes	79.65	—	37.87	—	79.98	—	21.3	12.1
Floods	98.40	36.97	1.44	44.95	99.40	—	28.8	15.9
Load E	70.41	—	28.63	—	70.74	—	5.7	3.5

Safety Factor

Types / Cases	Q_{ca}	Q_{ta}	SF_c	SF_t	SF_a
Oblique Wind	330	74	2.82	5.08	2.00
Oblique Wind	〃	〃	2.82	5.08	2.00
Earthquakes	〃	〃	4.12	Non Tension	1.33
Floods	〃	〃	3.31	1.64	1.33
Load E	〃	〃	4.66	Non Tension	2.00

→ O.K

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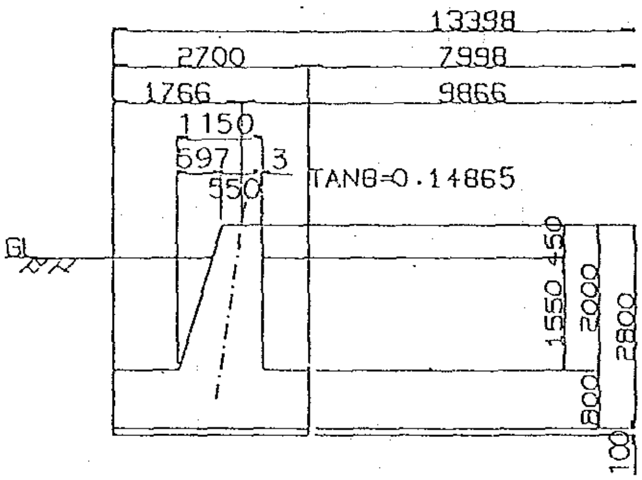
(6). B-D-I

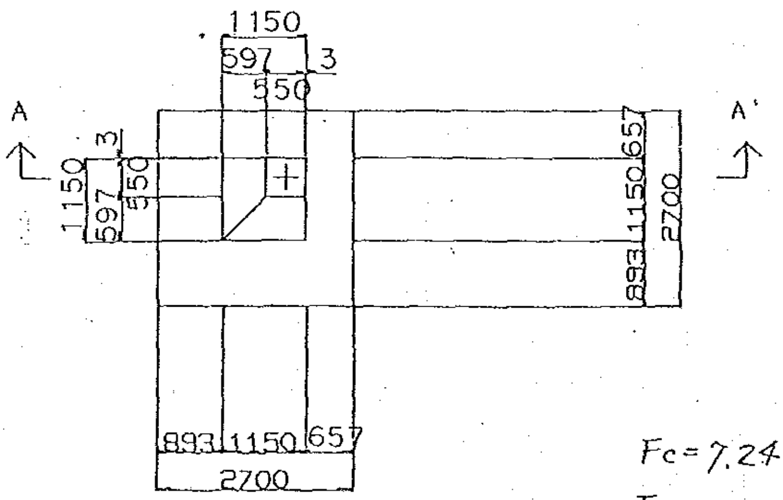
$C = 55.84^t$
$T = 41.65$
$Q = 4.13$
$Q_B = 1.49$

S=1/100

PROFILE



A-A' SECTION



$Fc = 7.24 > 2.00$
$Ft = 2.07 > 2.00$

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

------------------------------INPUT DATA------------------------------

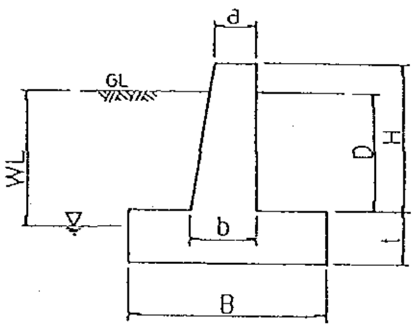
TOWER NAME............NO. B-D-I

(1)LOAD CONDITION

Comp. Load...................C = 55.84 TON
Uplift Load..................T = 41.65 TON
Hori. Load...................Q = 4.13 TON
Hori. Load...................QB= 1.49 TON
E-Load.......................TO= 9.85 TON

(2)DIMENSION OF FOUNDATION

a = 0.550 M
b = 1.150 M
B = 2.700 M
t = 0.800 M
H = 2.000 M
D = 1.550 M
WL=- 10.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation)................C1= 0.000 TON/M^2
Cohesion (on the foundation).....................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation).... φ = 40.000 °
Bearing Capacity Factor..N1= 95.700 N2=114.000 N3= 83.200 (φ =40 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

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---------------------------------RESULT-----------------------------------

(5)VERTICAL LOAD

```
Weight of concrite.............................WC=   17.609 TON
Weight of Soil.................................WS=   17.940 TON
Weight of Column and a part under Column...WC'=    6.151 TON
Vertical Load.......................P=C+WC+WS=   91.389 TON
```

(6)PILE CAPACITY

```
Comp. Side.....................................QCY= 100.000 TON/M^2
Uplift Side....................................QTY=  86.588 TON/M^2
Hori. Side.....................................QHY=  40.000 TON/M^2
```

Upper Limit

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.046 M <= B/6=  0.450 M
Incremental Rate...................MU=1+6*E/B=  1.101
Max. Soil Reaction.................Qmax=P/A*MU= 13.808 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=  0.084 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.864
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  1.912 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side......................SFC=QCY/Qmax=  7.242 >=  2.00 =OK=
Uplift Side.........................SFT=QTY/T=  2.078 >=  2.00 =OK=
Hori. Side.....................SFH=QHY/QHmax= 20.920 >=  2.00 =OK=
```

(10)E-LOAD

From Upper Limit

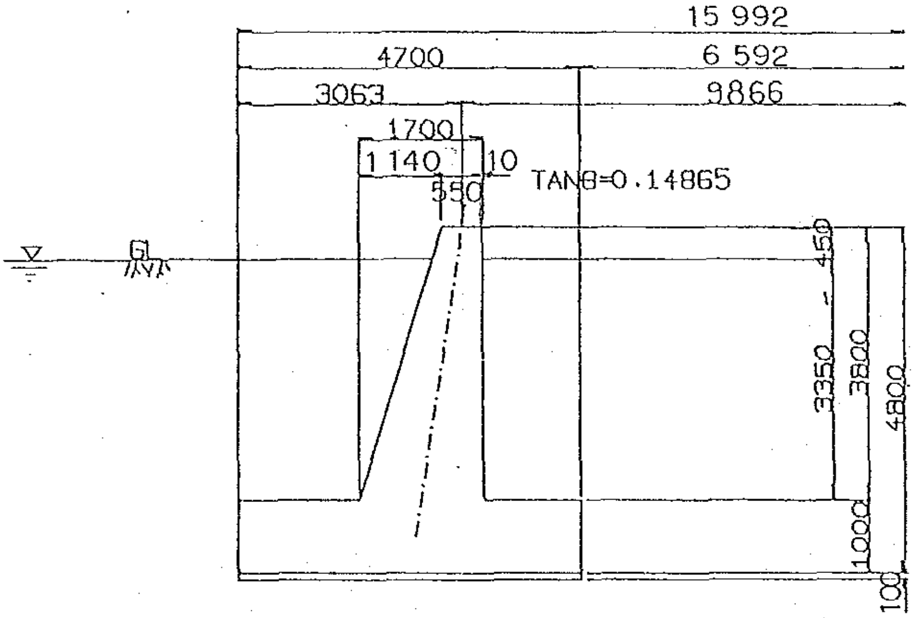
```
WC+WS'=   31.961 TON >= TO=    9.850 TON =OK=
   (WS'=WS*0.8)
```

(7). B- C -I

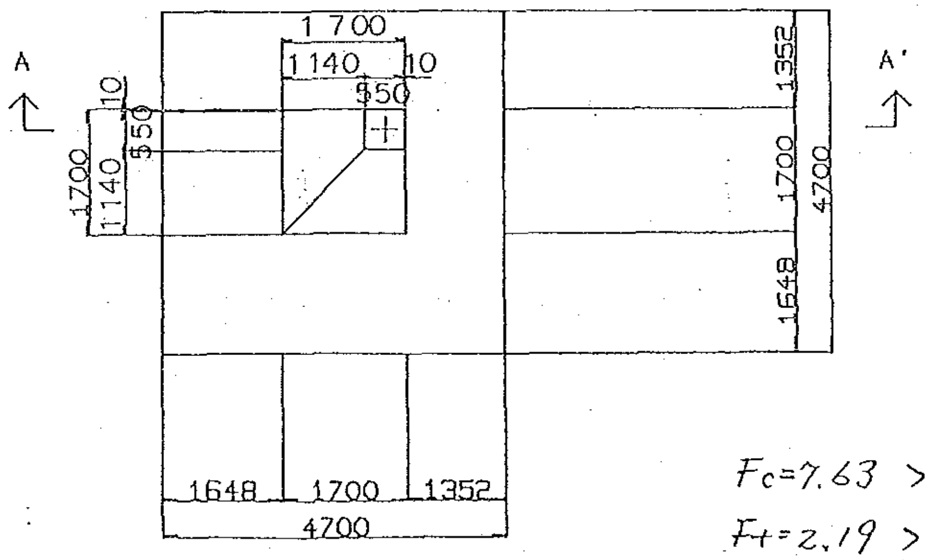
C= 55.84[+]
T= 41.65
Q= 4.13
Q_A= 1.49

S=1/100

PROFILE



A-A' SECTION



Fc=7.63 > 2.00
Ft=2.19 > 2.00

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

------------------------------INPUT DATA------------------------------

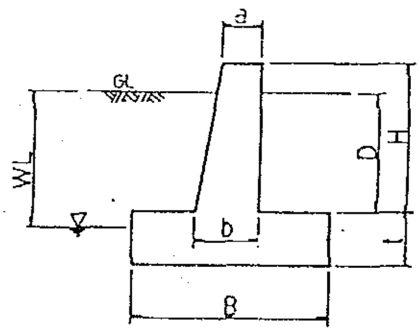
TOWER NAME............NO. B-C-I

(1)LOAD CONDITION

```
Comp. Load....................C =  55.84 TON
Uplift Load...................T =  41.65 TON
Hori. Load....................Q =   4.13 TON
Hori. Load....................QB=   1.49 TON
E-Load........................TO=   9.85 TON
```

(2)DIMENSION OF FOUNDATION

```
a  =   0.550 M
b  =   1.700 M
B  =   4.700 M
t  =   1.000 M
H  =   3.800 M
D  =   3.350 M
WL=-   0.000 M
γ  =   2.40 TON/M^3
```



(3)SOIL CONDITION

```
Unit Weight of Soil (beneath the foundation)......S1=  1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2=  1.440 TON/M^3
Cohesion (beneath the foundation)..................C1=  0.000 TON/M^2
Cohesion (on the foundation).......................C2=  0.000 TON/M^2
Angle of Internal Friction (on the foundation)....φ = 35.000 °
Bearing Capacity Factor..N1= 35.400 N2= 23.400 N3= 27.800 (φ=35 °)
```

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

104

------------------------------------RESULT------------------------------------

(5)VERTICAL LOAD

```
Weight of concrite.........................WC=   65.564 TON
Weight of Soil.............................WS=  124.103 TON
Weight of Column and a part under Column...WC'=  11.538 TON
Vertical Load......................P=C+WC+WS=  245.506 TON
```

(6)PILE CAPACITY

```
Comp. Side................................QCY=  87.958 TON/M^2
Uplift Side...............................QTY=  91.537 TON/M^2
Hori. Side................................QHY=  35.183 TON/M^2
```

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.029 M <= B/6=  0.783 M
Incremental Rate...................MU=1+6*E/B=  1.037
Max. Soil Reaction................Qmax=P/A*MU= 11.527 TON/M^2
```

2)Uplift Side

```
Eccentricity..................E=QB*H/(T-WC')=  0.188 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.825
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  0.879 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side.......................SFC=QCY/Qmax=  7.630 >=  2.00 =OK=
Uplift Side.........................SFT=QTY/T=  2.197 >=  2.00 =OK=
Hori. Side......................SFH=QHY/QHmax= 40.039 >=  2.00 =OK=
```

(10)E-LOAD

```
WC+WS'=   68.754 TON >= TO=    9.850 TON =OK=
   (WS'=WS*0.8)
```

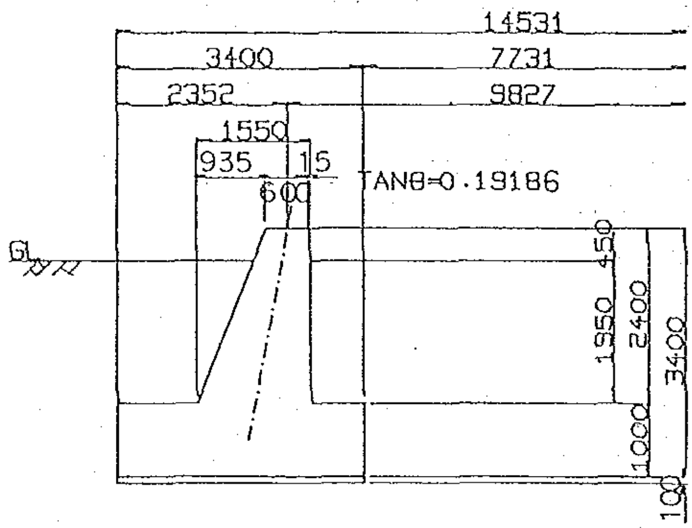

405

(8). D-D-I

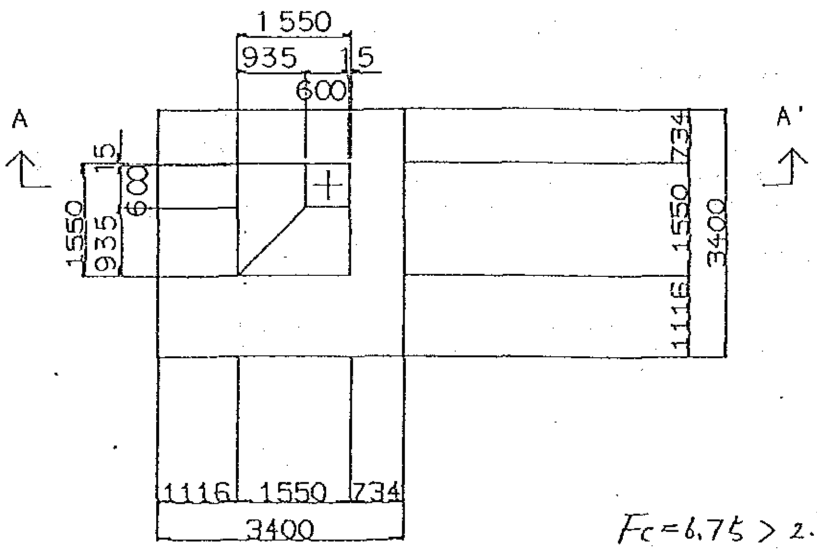
C= 95.63[t]
T= 80.88
Q= 3.58
Q_8= 0.84

S=1/100

PROFILE



A-A' SECTION



Fc=6.75 > 2.00

Ft= 2.02 > 2.00

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

-------------------------------INPUT DATA--------------------------------------

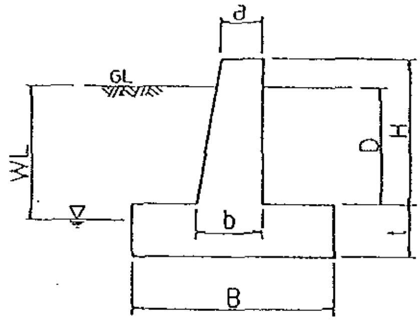
TOWER NAME............NO. D-D-I,C-D-I

(1)LOAD CONDITION

Comp. Load...................C = 95.63 TON
Uplift Load..................T = 80.88 TON
Hori. Load...................Q = 3.58 TON
Hori. Load...................QB= 0.84 TON
E-Load.......................TO= 34.04 TON

(2)DIMENSION OF FOUNDATION

a = 0.600 M
b = 1.550 M
B = 3.400 M
t = 1.000 M
H = 2.400 M
D = 1.950 M
WL=- 10.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation)................C1= 0.000 TON/M^2
Cohesion (on the foundation).....................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation)....ϕ = 40.000 °
Bearing Capacity Factor..N1= 95.700 N2=114.000 N3= 83.200 (ϕ =40 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

------------------------------------RESULT------------------------------------

(5)VERTICAL LOAD

```
Weight of concrite..........................WC=   34.834 TON
Weight of Soil..............................WS=   35.645 TON
Weight of Column and a part under Column...WC'=   12.856 TON
Vertical Load.......................P=C+WC+WS=  166.109 TON
```

(6)PILE CAPACITY

```
Comp. Side..................................QCY= 100.000 TON/M^2
Uplift Side.................................QTY= 163.487 TON/M^2
Hori. Side..................................QHY=  40.000 TON/M^2
```

Upper Limit

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.017 M <= B/6=  0.567 M
Incremental Rate....................MU=1+6*E/B=  1.030
Max. Soil Reaction................Qmax=P/A*MU= 14.805 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=  0.030 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.958
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  1.053 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side.......................SFC=QCY/Qmax=  6.754 >=  2.00 =OK=
Uplift Side..........................SFT=QTY/T=  2.021 >=  2.00 =OK=
Hori. Side.......................SFH=QHY/QHmax= 37.986 >=  2.00 =OK=
```

(10)E-LOAD

From Upper Limit

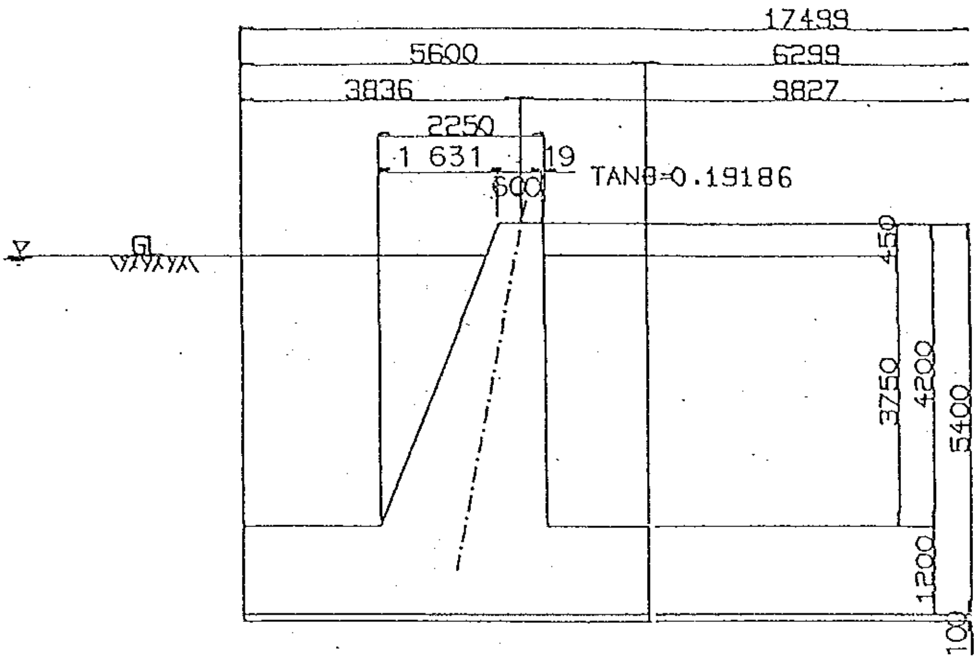
```
WC+WS'=   63.350 TON >= TO=   34.040 TON =OK=
   (WS'=WS*0.8)
```

(9). D-C-I, C-C-I

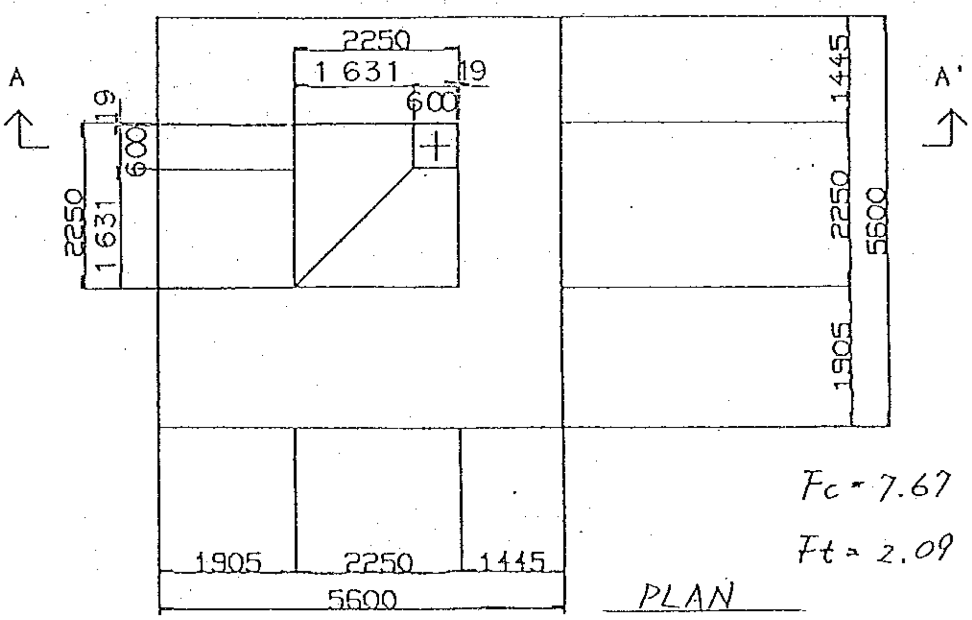
$C = 95.63^{t}$
$T = 80.88$
$Q = 3.58$
$Q_{B} = 0.84$

PROFILE

S=1/100



A-A' SECTION



$F_c = 7.67 > 2.00$

$F_t = 2.09 > 2.00$

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

------------------------------INPUT DATA------------------------------------

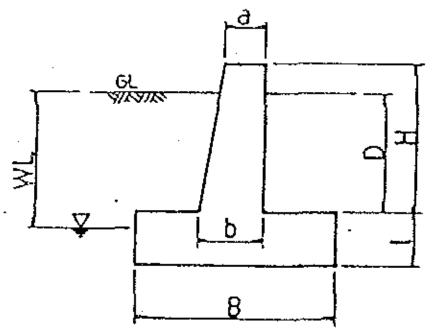
TOWER NAME............NO. D-C-I, C-C-I

(1)LOAD CONDITION

Comp. Load....................C = 95.63 TON
Uplift Load...................T = 80.88 TON
Hori. Load....................Q = 3.58 TON
Hori. Load....................QB= 0.84 TON
E-Load........................TO= 34.04 TON

(2)DIMENSION OF FOUNDATION

a = 0.600 M
b = 2.250 M
B = 5.600 M
t = 1.200 M
H = 4.200 M
D = 3.750 M
WL=- 0.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation).................C1= 0.000 TON/M^2
Cohesion (on the foundation)......................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation).... φ = 35.000 °
Bearing Capacity Factor..N1= 35.400 N2= 23.400 N3= 27.800 (φ =35 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

2/0

-----------------------------------RESULT---

(5)VERTICAL LOAD

```
Weight of concrite..........................WC=  113.072 TON
Weight of Soil..............................WS=  194.999 TON
Weight of Column and a part under Column...WC'=   21.994 TON
Vertical Load.......................P=C+WC+WS=  403.702 TON
```

(6)PILE CAPACITY

```
Comp. Side................................QCY= 100.000 TON/M^2
Uplift Side...............................QTY= 169.594 TON/M^2
Hori. Side................................QHY=  40.000 TON/M^2
```

Upper Limi

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.011 M <= B/6=  0.933 M
Incremental Rate...................MU=1+6*E/B=  1.012
Max. Soil Reaction...............Qmax=P/A*MU= 13.028 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=  0.060 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.948
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  0.533 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side.......................SFC=QCY/Qmax=  7.675 >=  2.00 =OK=
Uplift Side.........................SFT=QTY/T=  2.096 >=  2.00 =OK=
Hori. Side......................SFH=QHY/QHmax= 75.046 >=  2.00 =OK=
```

(10)E-LOAD

From Upper Limit

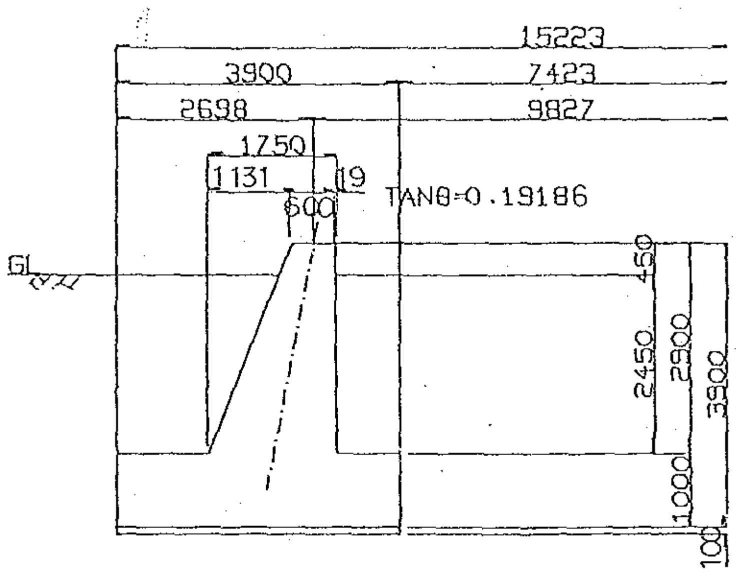
```
WC+WS'=  113.840 TON >= TO=   34.040 TON =OK=
   (WS'=WS*0.8)
```

(10). DR-D-I

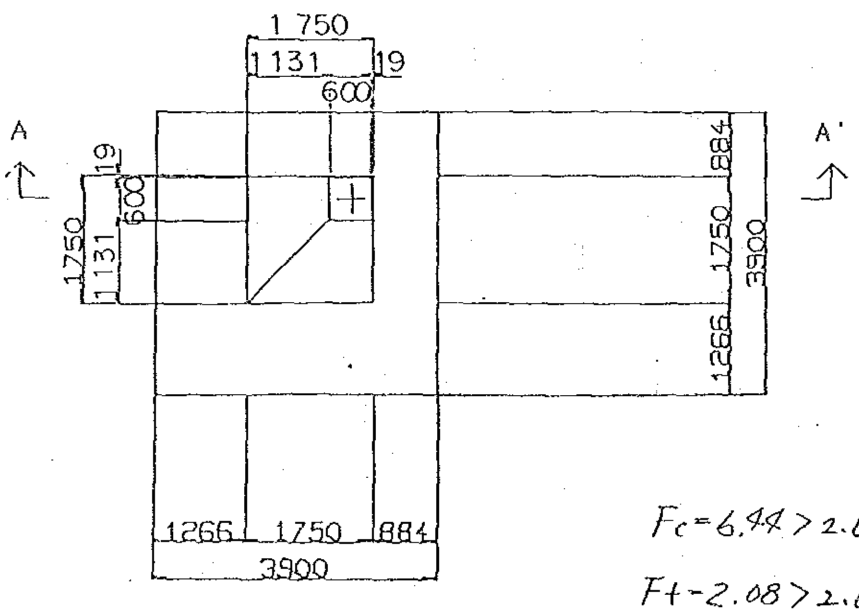
$C = 124.05^{t}$
$T = 107.47$
$Q = 3.67$
$Q_B = 0.87$

PROFILE

S=1/100



A - A' SECTION



$F_c = 6.44 > 2.00$

$F_t = 2.08 > 2.00$

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

----------------------------INPUT DATA------------------------------------

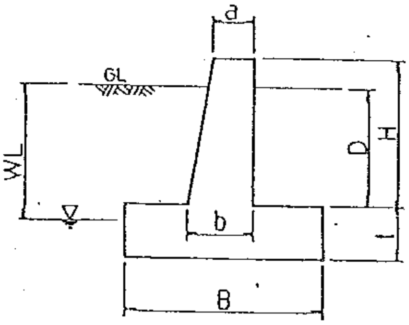
TOWER NAME............NO. DR-D-I

(1)LOAD CONDITION

Comp. Load...................C = 124.05 TON
Uplift Load..................T = 107.47 TON
Hori. Load...................Q = 3.67 TON
Hori. Load...................QB= 0.87 TON
E-Load......................TO= 48.35 TON

(2)DIMENSION OF FOUNDATION

a = 0.600 M
b = 1.750 M
B = 3.900 M
t = 1.000 M
H = 2.900 M
D = 2.450 M
WL=- 10.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation).................C1= 0.000 TON/M^2
Cohesion (on the foundation)......................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation)....ϕ = 40.000 °
Bearing Capacity Factor..N1= 95.700 N2=114.000 N3= 83.200 (ϕ =40 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

2/3

-----------------------------------RESULT-----------------------------------

(5)VERTICAL LOAD

```
Weight of concrite.........................WC=    46.880 TON
Weight of Soil.............................WS=    59.681 TON
Weight of Column and a part under Column...WC'=   17.726 TON
Vertical Load..........................P=C+WC+WS= 230.611 TON
```

(6)PILE CAPACITY

```
Comp. Side...............................QCY= 100.000 TON/M^2
Uplift Side..............................QTY= 224.264 TON/M^2
Hori. Side...............................QHY=  40.000 TON/M^2
```

(7)SOIL REACTION   Upper Limit

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=   0.015 M <= B/6=  0.650 M
Incremental Rate...................MU=1+6*E/B=   1.023
Max. Soil Reaction................Qmax=P/A*MU=  15.505 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=   0.028 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=   0.965
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=   0.941 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side......................SFC=QCY/Qmax=   6.449  >=  2.00 =OK=
Uplift Side.......................SFT=QTY/T=   2.086  >=  2.00 =OK=
Hori. Side.....................SFH=QHY/QHmax=  42.507  >=  2.00 =OK=
```

(10)E-LOAD   From Upper Limit

```
WC+WS'=   94.625 TON >= TO=   48.350 TON =OK=
   (WS'=WS*0.8)
```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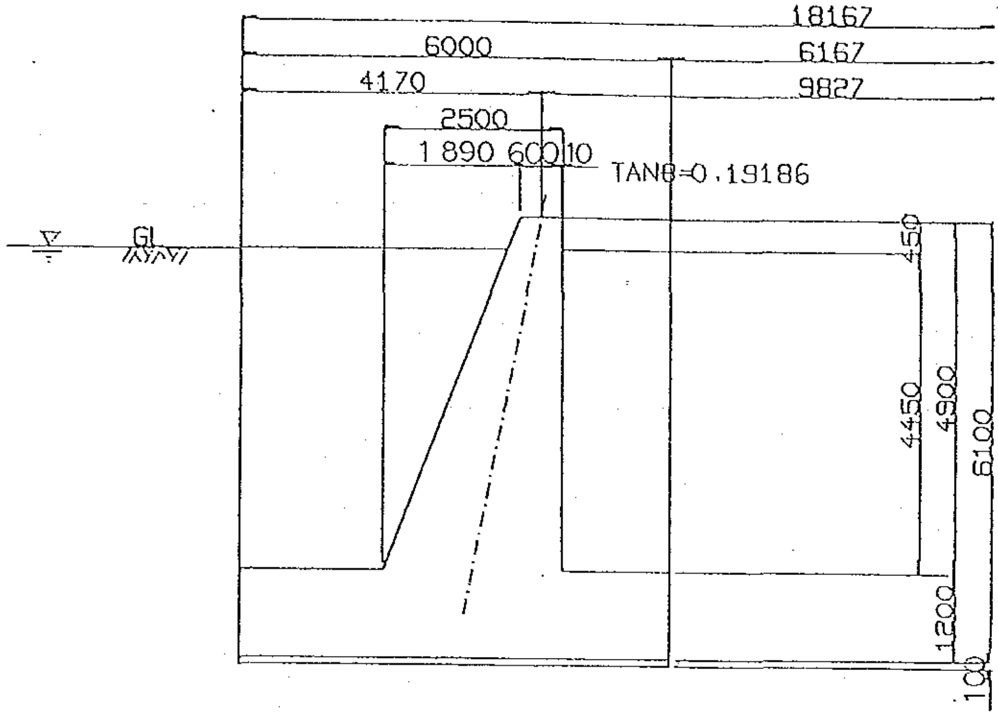
(11). DR- .C -I

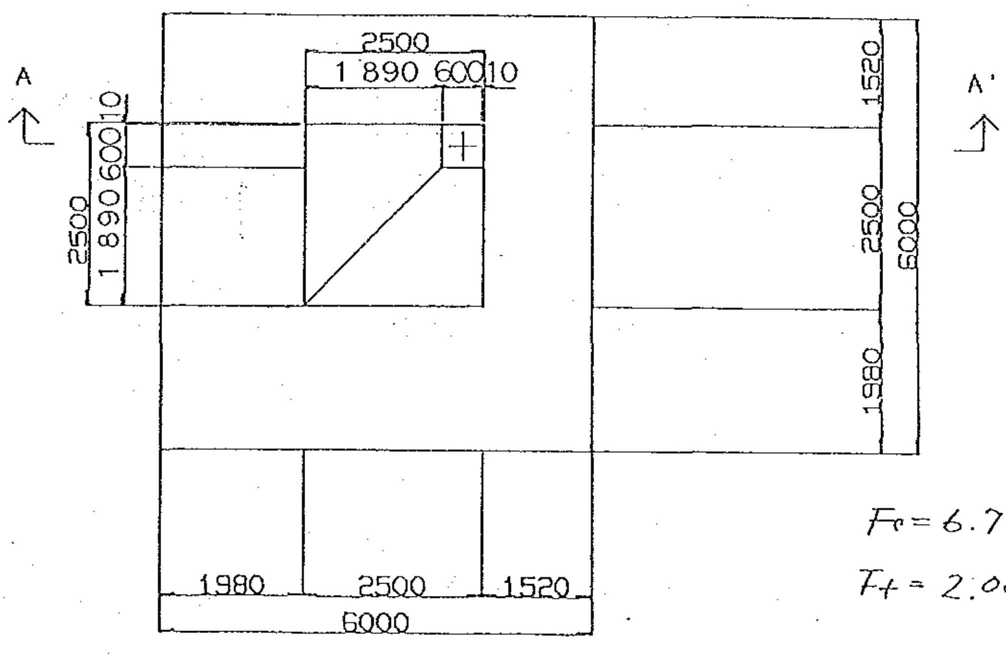
PROFILE

$C = 124.05^t$
$T = 107.47$
$Q = 3.67$
$Q_B = 0.87$

S=1/100



A - A' SECTION



$F_c = 6.79 > 2.00$

$F_t = 2.08 > 2.00$

PLAN

INDIVIDUAL TYPE TOWER FOUNDATION (I , II)

---------------------------------INPUT DATA---------------------------------------

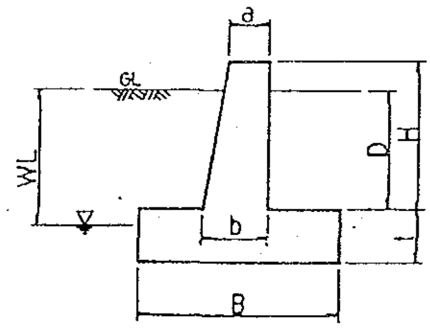
TOWER NAME............NO. DR-C-I

(1)LOAD CONDITION

Comp. Load......................C = 124.05 TON
Uplift Load.....................T = 107.47 TON
Hori. Load......................Q = 3.67 TON
Hori. Load......................QB= 0.87 TON
E-Load..........................TO= 48.35 TON

(2)DIMENSION OF FOUNDATION

a = 0.600 M
b = 2.500 M
B = 6.000 M
t = 1.200 M
H = 4.900 M
D = 4.450 M
WL=- 0.000 M
γ = 2.40 TON/M^3



(3)SOIL CONDITION

Unit Weight of Soil (beneath the foundation)......S1= 1.800 TON/M^3
Unit Weight of Soil (on the foundation)...........S2= 1.440 TON/M^3
Cohesion (beneath the foundation)................C1= 0.000 TON/M^2
Cohesion (on the foundation).....................C2= 0.000 TON/M^2
Angle of Internal Friction (on the foundation)....φ = 35.000 °
Bearing Capacity Factor..N1= 35.400 N2= 23.400 N3= 27.800 (φ =35 °)

(4)FOUNDATION TYPE

Foundation Type (shallow type...1,deep type...2)... 2

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-----------------------------------RESULT-----------------------------------

(5)VERTICAL LOAD

```
Weight of concrite..............................WC=  135.471 TON
Weight of Soil..................................WS=  264.901 TON
Weight of Column and a part under Column...WC'=   29.259 TON
Vertical Load.......................P=C+WC+WS=  524.422 TON
```

(6)PILE CAPACITY

```
Comp. Side...................................QCY= 100.000 TON/M^2
Uplift Side..................................QTY= 224.478 TON/M^2
Hori. Side...................................QHY=  40.000 TON/M^2
```

Upper Limit

(7)SOIL REACTION

1)Comp. Side

```
Eccentricity....E=QB*(H+T')/P=  0.010 M <= B/6=  1.000 M
Incremental Rate...................MU=1+6*E/B=  1.010
Max. Soil Reaction................Qmax=P/A*MU= 14.715 TON/M^2
```

2)Uplift Side

```
Eccentricity...................E=QB*H/(T-WC')=  0.055 M
Reduction Rate.........K=1/(1+6*E*B/(B^2+b^2))=  0.956
```

(8)HORIZONTAL SOIL REACTION

```
Max. Horizontal soil Reaction....QHmax=Q/(B*t)=  0.510 TON/M^2
```

(9)SAFETY FACTOR

```
Comp. Side........................SFC=QCY/Qmax=  6.795 >=  2.00 =OK=
Uplift Side.........................SFT=QTY/T=  2.088 >=  2.00 =OK=
Hori. Side......................SFH=QHY/QHmax= 78.431 >=  2.00 =OK=
```

From Upper Limit

(10)E-LOAD

```
WC+WS'=  143.992 TON >= TO=   48.350 TON =OK=
   (WS'=WS*0.8)
```

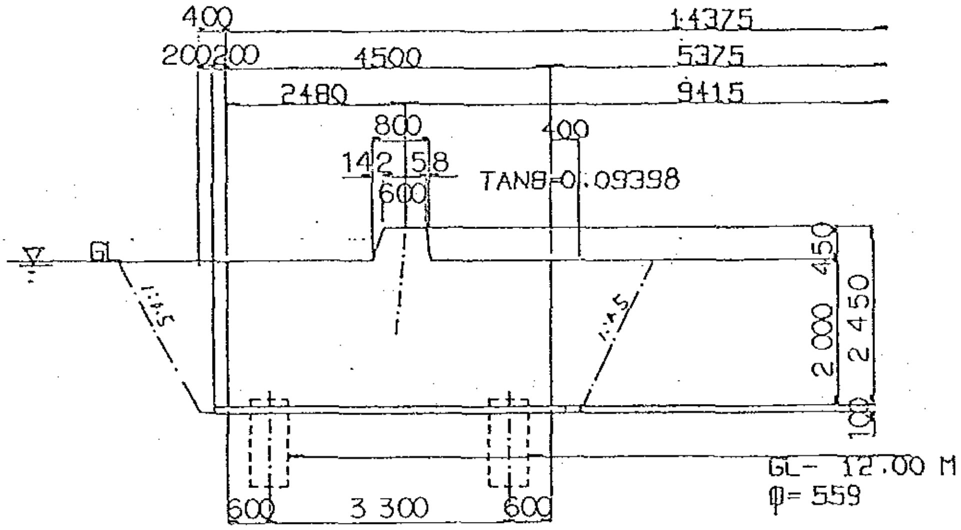

Ł/

(12). A4- S -Ⅲ

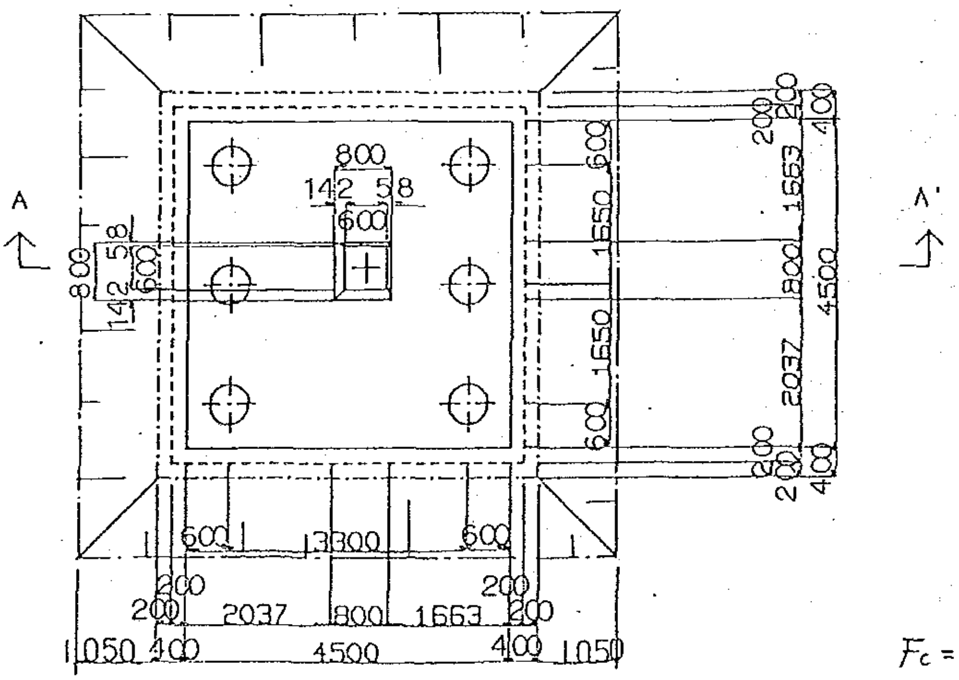
$C = 110.95^t$
$T = 84.56$
$Q_s = 2.34$

PROFILE

S=1/100



A-A' SECTION



$Fc = 2.40 > 2.00$
$Ft = 2.04 > 2.00$

PLAN

TLG-1-236

PILE TYPE TOWER FOUNDATION (Ⅲ)

----------------------------------INPUT DATA----------------------------------

TOWER NAME.....NO. A4-S-Ⅲ

(1)LOAD CONDITION

Comp. Load..........................C =	110.95	TON
Uplift Load.........................T =	84.56	TON
Hori. Load..........................Q =	9.24	TON
Hori. Load..........................QB=	2.34	TON
E-Load..............................TO=	7.60	TON

(2)STRATUM DATA

No.	Stratum No.	N-Value(N)	Thickness(L)	N*L
1	1	0(0)	2.00 M	0.000(0.000)
2	1	1(1)	1.00 M	1.000(1.000)
3	2	3(3)	1.00 M	2.500(2.500)
4	1	7(7)	6.50 M	45.500(45.500)
5	2	16(6)	1.50 M	24.000(9.000)

Sand (total)....................NS*LS= 46.500 TON
(46.500)

Clay (total)....................QU*LC= 26.500 TON
(11.500)

(3)WATER TABLE......................WL=- 0.000

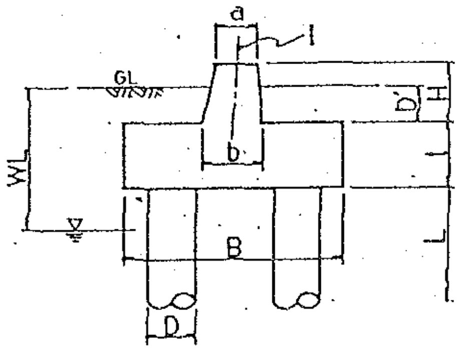
(4)N-VALUE OF BEARING STRATUM.........N = 29

(5)BULK DENSITY

Concrete...........................γ = 2.400 TON/M^3
Soil (on the foundation)...........S2= 0.000 TON/M^3

(6)DIMENSION OF FOUNDATION

a = 0.600 M
b = 0.700 M
B = 4.500 M
t = 2.000 M
H = 0.450 M
D'= 0.000 M
I = 0.09398



(7)PILE TYPE (driven pile...1,cast in place conc. pile...2).... 2

(8)PILE CONDITION

Diameter of Pile....................D = 0.559 M
Length of Pile......................L = 12.000 M
Unit Weight (without buoyancy)......G1= 0.589 TON/M
Unit Weight (with buoyancy).........G2= 0.344 TON/M
Blockade Ratio......................R = 1.000

(9)LIMIT OF PILE CAPACITY

Comp. Side.........................QCA= 124.000 TON/UNIT
Uplift Side........................QTA= 49.000 TON/UNIT

(10)CONFIGURATION

Row of pile	Number(UNIT)	Distance(M)
1	2	1.650
2	2	0.000
3	2	-1.650

Eccentricity.......................E= 0.000 M

------------------------------------RESULT------------------------------------

(11)VERTICAL LOAD

1)Without Buoyancy

```
Weight of Concrete...................WC=    97.657 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=  208.607 TON
```

2)With Buoyancy

```
Weight of Concrete...................WC=    57.157 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=  168.107 TON
```

(12)PILE CAPACITY

```
Comp. Side...QCY=15*R*N*π*D^2/4+((NS*LS)/5+(QU*LC)/2)*π*D)/1.5-2*WP
             =  85.793 TON/UNIT

Uplift Side..QTY=((NS*LS)/5+(QU*LC)/2)*π*D+1.5WP')/1.5
             =  21.060 TON/UNIT
```

(13)OVERTURNING MOMENT

1)Without Buoyancy

```
Comp. Side...............MC=QB*(H+t)+P*E=     5.733 TON-M
Uplift Side....MT=QB*(H+t)+|T-WC-WS|*E=     5.733 TON-M
```

2)With Buoyancy

```
Comp. Side...............MC=QB*(H+t)+P*E=     5.733 TON-M
Uplift Side....MT=QB*(H+t)+|T-WC-WS|*E=     5.733 TON-M
```

(14)WEIGHT OF PILE

```
Without Buoyancy.....................WP=     5.890 TON/UNIT
With Buoyancy.......................WP'=     3.440 TON/UNIT
```

(15)LOAD DISTRIBUTIVE

Without Buoyancy......................KA= 0.436
With Bouyancy.........................KA= 0.311

(16)PILE REACTION

1)Comp. Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	35.637	28.887
2	34.768	28.018
3	33.899	27.149

2)Uplift Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	8.440	10.302
2	7.950	9.704
3	7.460	9.106

(17)SAFETY FACTOR

1)Without Buoyancy

Comp. Side................SFC=QCY/NCmax= 2.407 >= 2.0 =OK=
Uplift Side...............SFT=QTY/NTmax= 2.495 >= 2.0 =OK=

2)With Buoyancy

Comp. Side................SFC=QCY/NCmax= 2.970 >= 2.0 =OK=
Uplift Side...............SFT=QTY/NTmax= 2.044 >= 2.0 =OK=

(18)E-LOAD

1)Without Buoyancy

WC+WS+N*WP= 132.997 TON >TO= 7.600 TON =OK=

2)With Buoyancy

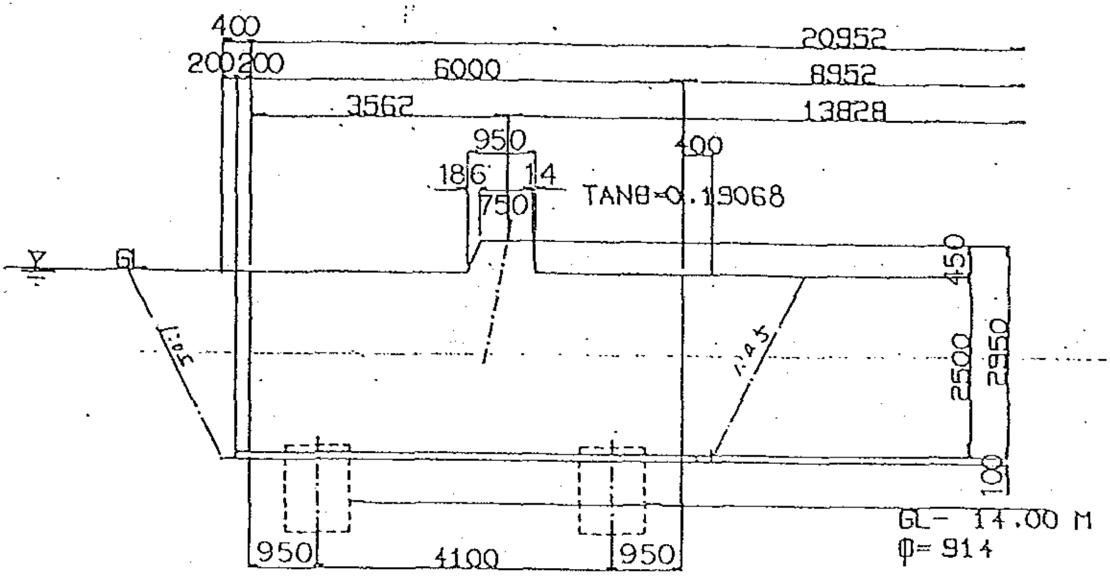
WC+WS+N*WP'= 77.797 TON >TO= 7.600 TON =OK=

(13) DR4-S-Ⅲ, D4-'S-Ⅲ

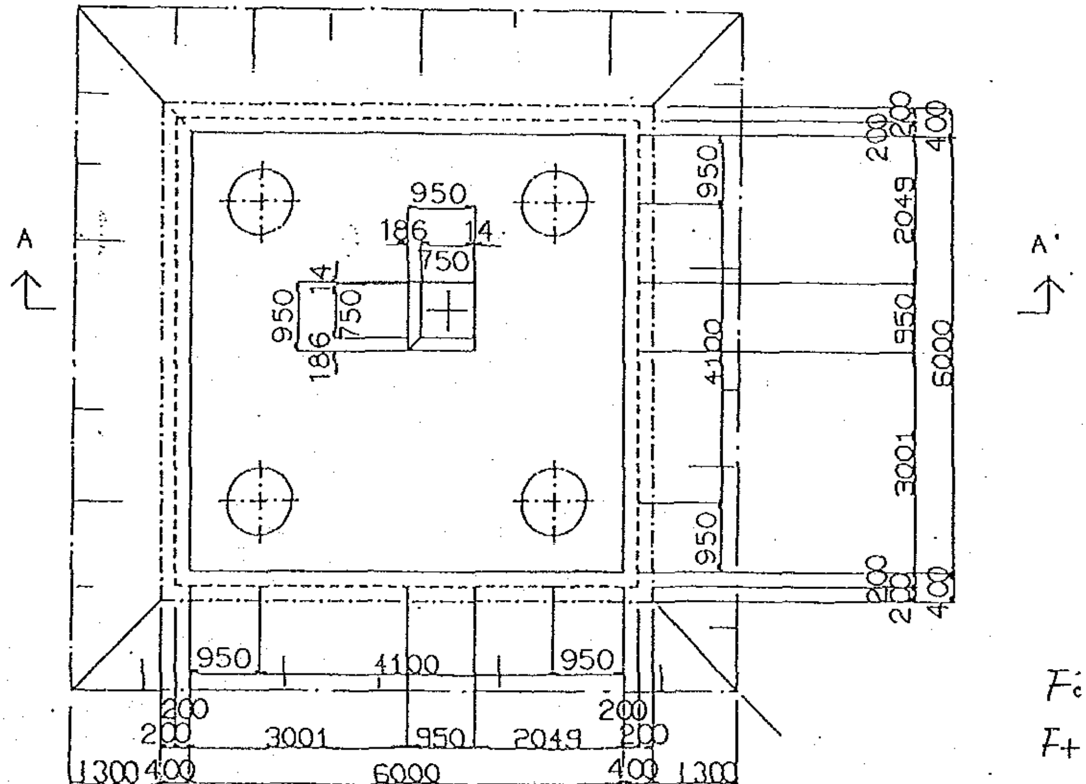
C = 245.32[t]
T = 201.88
Q_s = 0.78

S=1/100

PROFILE



A-A' SECTION



F_c = 2.88 > 2.00
F_t = 2.01 > 2.00

PLAN

PILE TYPE TOWER FOUNDATION (Ⅲ)

----------------------------------INPUT DATA----------------------------------

TOWER NAME.....NO. DR4-S-Ⅲ, D4-S-Ⅲ

(1)LOAD CONDITION

Comp. Load..........................C = 245.32 TON
Uplift Load.........................T = 201.88 TON
Hori. Load..........................Q = 12.37 TON
Hori. Load..........................QB= 0.78 TON
E-Load..............................TO= 76.67 TON

(2)STRATUM DATA

No.	Stratum No.	N-Value(N)	Thickness(L)	N*L
1	1	0(0)	2.50 M	0.000(0.000)
2	1	1(1)	0.50 M	0.500(0.500)
3	2	3(3)	1.00 M	2.500(2.500)
4	1	7(7)	6.50 M	45.500(45.500)
5	2	16(6)	2.00 M	32.000(12.000)
6	1	50(50)	1.50 M	75.000(75.000)

Sand (total)....................NS*LS= 121.000 TON
(121.000)
Clay (total)....................QU*LC= 34.500 TON
(14.500)

(3)WATER TABLE......................WL=- 0.000

(4)N-VALUE OF BEARING STRATUM.........N = 46

(5)BULK DENSITY

Concrete............................γ = 2.400 TON/M^3
Soil (on the foundation)............S2= 0.000 TON/M^3

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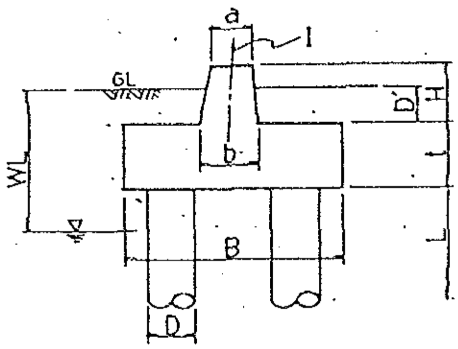
(6)DIMENSION OF FOUNDATION

a = 0.750 M
b = 0.950 M
B = 6.000 M
t = 2.500 M
H = 0.450 M
D'= 0.000 M
I = 0.19068



(7)PILE TYPE (driven pile...1,cast in place conc. pile...2).... 2

(8)PILE CONDITION

Diameter of Pile....................D = 0.914 M
Length of Pile......................L = 14.000 M
Unit Weight (without buoyancy)......G1= 1.575 TON/M
Unit Weight (with buoyancy).........G2= 0.919 TON/M
Blockade Ratio......................R = 1.000

(9)LIMIT OF PILE CAPACITY

Comp. Side........................QCA= 334.000 TON/UNIT
Uplift Side.......................QTA= 131.000 TON/UNIT

(10)CONFIGURATION

Row of pile	Number(UNIT)	Distance(M)
1	2	2.050
2	2	-2.050

Eccentricity.......................E= 0.000 M

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--RESULT--

(11)VERTICAL LOAD

1)Without Buoyancy

```
Weight of Concrete.....................WC=  216.784 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=  462.104 TON
```

2)With Buoyancy

```
Weight of Concrete.....................WC=  126.784 TON
Weight of Soil (on the foundation)....WS=    0.000 TON
Vertical Load.................P=C+WC+WS=  372.104 TON
```

(12)PILE CAPACITY

```
Comp. Side...QCY=15*R*N*π*D^2/4+((NS*LS)/5+(QU*LC)/2)*π*D)/1.5-2*WP
             = 334.000 TON/UNIT

Uplift Side..QTY=((NS*LS)/5+(QU*LC)/2)*π*D+1.5WP')/1.5
             =  70.773 TON/UNIT
```

(13)OVERTURNING MOMENT

1)Without Buoyancy

```
Comp. Side...............MC=QB*(H+t)+P*E=    2.301 TON-M
Uplift Side....MT=QB*(H+t)+|T-WC-WS|*E=    2.301 TON-M
```

2)With Buoyancy

```
Comp. Side...............MC=QB*(H+t)+P*E=    2.301 TON-M
Uplift Side....MT=QB*(H+t)+|T-WC-WS|*E=    2.301 TON-M
```

(14)WEIGHT OF PILE

```
Without Buoyancy......................WP=   18.113 TON/UNIT
With Buoyancy.........................WP'=   10.569 TON/UNIT
```


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(15)LOAD DISTRIBUTIVE

Without Buoyancy......................KA= 0.434
With Bouyancy.........................KA= 0.309

(16)PILE REACTION

1)Comp. Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	115.807	93.307
2	115.245	92.745

2)Uplift Side

Row of pile	Without Buoyancy (TON/UNIT)	With Buoyancy (TON/UNIT)
1	28.741	35.052
2	28.423	34.665

(17)SAFETY FACTOR

1)Without Buoyancy

Comp. Side................SFC=QCY/NCmax= 2.884 >= 2.0 =OK=
Uplift Side...............SFT=QTY/NTmax= 2.462 >= 2.0 =OK=

2)With Buoyancy

Comp. Side................SFC=QCY/NCmax= 3.579 >= 2.0 =OK=
Uplift Side...............SFT=QTY/NTmax= 2.019 >= 2.0 =OK=

(18)E-LOAD

1)Without Buoyancy

WC+WS+N*WP= 289.234 TON >TO= 76.670 TON =OK=

2)With Buoyancy

WC+WS+N*WP'= 169.058 TON >TO= 76.670 TON =OK=

11. LEDGER FOR INDIVIDUAL TOWER

West Wharf P/S — b c / a d — Baldia G/S

LEDGER FOR INDIVIDUAL TOWER

Circuit Voltage : 220 kV
No. of Circuit : 2 cct.
Conductor : ACSR/AS 330 mm^2
Ground Wire : OPGW 190/90 mm^2

Tower No.	Span (m)	Wind Span (m)	Daviation Angle	Tower Type	Tower Extention	Ground Level Difference (m)	Formation Level (m)	Cond. Support Height (m)	Cond. Support Level Difference (m)	h1/S1	h2/S2	Σh/S	Catenary Ang. (°) For side	Catenary Ang. (°) Back side	Catenary Ang. (°) Total	Insulator Class	Insulator Type	Type of Foundation	Remarks: Crossing	Remarks: Land Item
G1	15			Gantry	+0	0.0	0.0	9.0								12t*1	—	PILE FOUND.		Factory Area
G2	35			Gantry	+0	-0.2	0.0	9.0								12t*1	—	PILE FOUND.		
1	320	185		D4	-3	-0.1	0.0	39.2	32.57	0.6514	-0.0102	0.6412	36	7	43	12t*1 12t*2	— =	D4-S		
2	330	325		A4	+0	0.0	0.0	42.5	3.28	0.0102	0.0095	0.0198	6	6	12	12t*2	V	A4-S		
3	276	303		A4	-3	-0.2	0.0	39.5	-3.15	-0.0095	0.0006	-0.0090	5	5	10	12t*2	V	A4-S	Road	
4	350	313	L 70° 48'	DR4	-3	0.1	0.0	39.2	-0.16	-0.0006	0.0556	0.0550	7	11	18	12t*2	= =	DR4-S	Road, Mosque	
5	380	365		A	+0	-2.9	0.0	22.5	-19.45	-0.0556	-0.0277	-0.0833	3	5	8	12t*2	V	A-S	Sea Water Houses, Boat Manuf.	Trees
6	500	440		AL	+6	0.7	0.0	32.5	10.53	0.0277	-0.0034	0.0243	8	8	16	12t*2	V	AL-S	Sea Water	Sea Water
7	460	480		AL	+7.5	0.1	0.0	34.0	1.72	0.0034	0.0175	0.0210	9	9	18	12t*2	V	AL-S	Sea Water, House	Enchroachment Area
8	430	445		AL	+0	-0.5	0.0	26.5	-8.06	-0.0175	0.0016	-0.0159	7	7	14	12t*2	V	AL-R	Fish Yard, River	River
9	333	382		A	+3	0.3	0.0	25.5	-0.70	-0.0016	0.0055	0.0039	7	6	13	12t*2	V	A-R	Layari River	River
10	290	312	L 36° 42'	C	+0	1.4	0.0	22.2	-1.84	-0.0055	0.0293	0.0238	8	9	17	12t*2	= =	C-C	Layari River, Bund	Water Lodging
11	225	258		AS	+0	-0.8	0.0	14.5	-8.50	-0.0293	-0.0024	-0.0317	3	4	7	12t*2	V	AS-C	Katcha Road Water Lodging	
12	220	223	L 44° 39'	D	-3	0.8	0.0	14.2	0.53	0.0024	0.0010	0.0033	6	6	12	12t*2	= =	D-C	ater Lodging	
13	220	220		AS	+0	-0.5	0.0	14.5	-0.21	-0.0010	-0.0059	-0.0068	4	3	7	12t*2	V	AS-C		Broken Area
14	220	220		AS	+0	1.3	0.0	14.5	1.29	0.0059	-0.0048	0.0011	4	3	7	12t*2	V	AS-C		
15	220	220		AS	+0	1.1	0.0	14.5	1.05	0.0048	0.0019	0.0066	4	4	8	12t*2	V	AS-C		
16	180	200		AS	+0	-0.8	0.0	14.5	-0.41	-0.0019	-0.0053	-0.0072	4	3	7	12t*2	V	AS-C		
17	170	175	R 4° 49'	B	-6	-0.6	0.0	16.2	1.11	0.0053	0.0163	0.0216	6	6	12	12t*2	= =	B-C	KMC Boundary, Market	
18																			H.T Line	Salt Area

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West Wharf P/S — Baldia G/S

LEDGER FOR INDIVIDUAL TOWER

Circuit Voltage : 220 kV
No. of Circuit : 2 cct.
Conductor : ACSR/AS 330 mm^2
Ground Wire : OPGW 190/90 mm^2

Tower No.	Span (m)	Wind Span (m)	Daviation Angle	Tower Type	Tower Extention	Ground Level Diffe-rence (m)	Forma-tion Level (m)	Cond. Support Height (m)	Cond. Support Level Diffe-rence (m)	h/S h1/S1	h/S h2/S2	h/S Σh/S	Calenary Ang. (°) For side	Calenary Ang. (°) Back side	Calenary Ang. (°) Total	Insulator Class	Insulator Type	Type of Foun-dation	Remarks Crossing	Remarks Land Item
18	245	208		AS	+0	-1.0	0.0	14.5	-2.74	-0.0161	0.0000	-0.0161	2	4	6	12t*2	V	AS-C	Old Salt Area	
19	245	245		AS	+0	0.0	0.0	14.5	0.00	0.0000	-0.0004	-0.0004	4	4	8	12t*2	V	AS-C	Old Salt Area Katcha Road	Old Salt Area
20	220	233		AS	+0	0.0	0.0	14.5	0.10	0.0004	0.0023	0.0027	4	4	8	12t*2	V	AS-C		
21	200	210		AS	+0	-0.5	0.0	14.5	-0.50	-0.0023	0.0000	-0.0023	3	3	6	12t*2	V	AS-C	Sea Water	Sea Water
22	200	200		AS	+0	0.0	0.0	14.5	0.00	0.0000	0.0005	0.0005	3	3	6	12t*2	V	AS-C	Sea Water, Nala	
23	230	215		AS	+0	-0.1	0.0	14.5	-0.10	-0.0005	0.0004	-0.0001	3	4	7	12t*2	V	AS-C	Sea Water	
24	220	225		AS	+0	-0.1	0.0	14.5	-0.10	-0.0004	0.0014	0.0009	4	4	8	12t*2	V	AS-C	Sea Water	
25	240	230		AS	+0	-0.3	0.0	14.5	-0.30	-0.0014	-0.0148	-0.0161	4	3	7	12t*2	V	AS-C	Bund, Sea Water	
26	180	210	L 12°27'	B	-6	1.8	0.0	16.2	3.54	0.0148	0.0180	0.0328	7	6	13	12t*2	= =	B-C	Sea Water	
27	180	180		AS	+0	-1.5	0.0	14.5	-3.21	-0.0178	-0.0051	-0.0230	2	3	5	12t*2	V	AS-C	Sea Water, Bushes	
28	180	180		AS	+0	0.9	0.0	14.5	0.92	0.0051	0.0067	0.0118	3	3	6	12t*2	V	AS-C	Sea Water	
29	160	170	R 71°22'	DR	-3	-1.0	0.0	14.2	-1.21	-0.0067	-0.0015	-0.0083	5	5	10	12t*2	=|=	DR-C	Marin Academy, Road	
30	240	200		AS	+0	-0.0	0.0	14.5	0.25	0.0015	-0.0013	0.0002	3	4	7	12t*2	V	AS-C	Sea Water	
31	240	240		AS	+0	0.3	0.0	14.5	0.32	0.0013	0.0038	0.0052	4	4	8	12t*2	V	AS-C	Sea Water	
32	240	240		AS	+0	-0.9	0.0	14.5	-0.92	-0.0038	-0.0037	-0.0075	4	4	8	12t*2	V	AS-C	Sea Water, Salt Area	Salt Area
33	250	245		AS	+0	0.9	0.0	14.5	0.88	0.0037	-0.0005	0.0032	4	4	8	12t*2	V	AS-C	Salt Area	
34	240	245		AS	+0	0.1	0.0	14.5	0.12	0.0005	0.0006	0.0011	4	4	8	12t*2	V	AS-C	Salt Area	
35	250	245		AS	+0	-0.1	0.0	14.5	-0.14	-0.0006	-0.0004	-0.0010	4	4	8	12t*2	V	AS-C		
36	180	215	R 52° 7'	D	-3	0.4	0.0	14.2	0.10	0.0004	-0.0004	-0.0000	6	5	11	12t*2	=|=	D-C		
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LEDGER FOR INDIVIDUAL TOWER

West Wharf P/S — b c / a d — Baldia G/S

Circuit Voltage : 220 kV
No. of Circuit : 2 cct.
Conductor : ACSR/AS 330 mm^2
Ground Wire : OPGW 190/90 mm^2

Tower No.	Span (m)	Wind Span (m)	Daviation Angle	Tower Type	Tower Extention	Ground Level Diffe-rence (m)	Forma-tion Level (m)	Cond. Support Height (m)	Cond. Support Level Diffe-rence (m)	h1/S1	h2/S2	Σh/S	Catenary Ang. (°) For side	Catenary Ang. (°) Back side	Catenary Ang. (°) Total	Insulator Class	Insulator Type	Type of Foun-dation	Remarks: Crossing	Remarks: Land Item
37	169	175		AS	+0	-0.2	0.0	14.5	0.08	0.0004	-0.0054	-0.0050	3	2	5	12t*2	V	AS-C		Salt Area
38	140	155	L 45°27'	D	-3	1.2	0.0	14.2	0.91	0.0054	0.0088	0.0142	5	5	10	12t*2	=|=	D-C	Road, H.T Line Old Salt Area	
39	220	180		AS	+0	-1.5	0.0	14.5	-1.23	-0.0088	0.0002	-0.0086	2	4	6	12t*2	V	AS-C	Sea Water	
40	240	230		AS	+0	-0.1	0.0	14.5	-0.05	-0.0002	-0.0020	-0.0022	4	4	8	12t*2	V	AS-C	Sea Water	Sea Water
41	240	240		AS	+0	0.5	0.0	14.5	0.47	0.0020	0.0019	0.0039	4	4	8	12t*2	V	AS-C	Sea Water	
42	240	240		AS	+0	-0.5	0.0	14.5	-0.46	-0.0019	-0.0011	-0.0030	4	4	8	12t*2	V	AS-C	Sea Water	
43	240	240		AS	+0	0.3	0.0	14.5	0.27	0.0011	-0.0034	-0.0023	4	4	8	12t*2	V	AS-C		Open Area
44	155	198		AS	+0	0.8	0.0	14.5	0.82	0.0034	0.0058	0.0093	4	3	7	12t*2	V	AS-C		
45	150	153		B	-6	-0.1	0.0	14.5	-0.91	-0.0058	-0.0084	-0.0142	2	2	4	12t*2	==	B-C	Nala, Road, H.T Line	
46	230	190		B	-6	1.6	0.0	14.5	1.04	0.0084	-0.0055	0.0029	3	3	6	12t*2	==	B-C		
47	250	240		AS	+0	1.0	0.0	14.5	1.26	0.0055	0.0001	0.0056	4	4	8	12t*2	V	AS-C	Road under const.	
48	250	250		AS	+0	-0.2	0.0	14.5	-0.02	-0.0001	-0.0318	-0.0319	4	2	6	12t*2	V	AS-C		
49	230	240	R 38°18'	C	+0	-0.1	0.0	22.2	7.64	0.0318	0.0311	0.0630	8	8	16	12t*2	==	C-C	Road under const.	
50	230	230		AS	+0	0.6	0.0	14.5	-7.16	-0.0311	-0.0043	-0.0354	2	4	6	12t*2	V	AS-D		
51	230	230		AS	+0	1.0	0.0	14.5	0.99	0.0043	-0.0040	0.0003	4	4	8	12t*2	V	AS-D	Trees & Bushies	
52	196	213		AS	+0	0.9	0.0	14.5	0.91	0.0040	-0.0046	-0.0007	4	3	7	12t*2	V	AS-D	Trees & Bushies	Bushes
53	224	210		AS	+0	0.9	0.0	14.5	0.97	0.0046	-0.0058	-0.0012	4	3	7	12t*2	V	AS-D	Road under const.	Bushes
54	210	217		AS	+0	1.3	0.0	14.5	1.22	0.0058	-0.0031	0.0027	4	3	7	12t*2	V	AS-D		Open Area
55	210	210		AS	+0	0.7	0.0	14.5	0.66	0.0031	-0.0055	-0.0024	4	3	7	12t*2	V	AS-D	Bushies	Bushies
56																				Bushies

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West Wharf P/S — b c / a d — Baldia G/S

LEDGER FOR INDIVIDUAL TOWER

Circuit Voltage : 220 kV
No. of Circuit : 2 cct.
Conductor : ACSR/AS 330 mm^2
Cround Wire : OPGW 190/90 mm^2

Tower No.	Span (m)	Wind Span (m)	Daviation Angle	Tower Type	Tower Extention	Ground Level Diffe-rence (m)	Forma-tion Level (m)	Cond. Support Height (m)	Cond. Support Level Diffe-rence (m)	h1/S1	h2/S2	Σh/S	Catenary Ang. (°) For side	Catenary Ang. (°) Back side	Catenary Ang. (°) Total	Insulator Class	Insulator Type	Type of Foun-dation	Remarks Crossing	Remarks Land Item
56	200	205		AS	+0	1.2	0.0	14.5	1.16	0.0055	-0.0048	0.0008	4	3	7	12t*2	V	AS-D		
57	200	200		AS	+0	1.0	0.0	14.5	0.95	0.0048	-0.0031	0.0017	4	3	7	12t*2	V	AS-D	Bushies	
58	215	208		AS	+0	0.6	0.0	14.5	0.61	0.0031	-0.0041	-0.0010	3	3	6	12t*2	V	AS-D		
59	200	208	R 56°51'	D	-3	1.1	0.0	14.2	0.87	0.0041	-0.0063	-0.0023	6	5	11	12t*2	==	D-D		Open Area
60	240	220		AS	+0	1.0	0.0	14.5	1.27	0.0063	-0.0035	0.0028	4	4	8	12t*2	V	AS-D	Road	
61	210	225		AS	+0	0.9	0.0	14.5	0.85	0.0035	-0.0057	-0.0021	4	3	7	12t*2	V	AS-D		
62	210	210		AS	+0	1.2	0.0	14.5	1.19	0.0057	-0.0049	0.0008	4	3	7	12t*2	V	AS-D		
63	210	210		AS	+0	1.0	0.0	14.5	1.02	0.0049	-0.0072	-0.0024	4	3	7	12t*2	V	AS-D		
64	210	210		AS	+0	1.5	0.0	14.5	1.52	0.0072	-0.0075	-0.0003	4	3	7	12t*2	V	AS-D		
65	210	210		AS	+0	1.6	0.0	14.5	1.58	0.0075	-0.0001	0.0074	4	4	8	12t*2	V	AS-D		Open Area
66	210	210		AS	+0	0.0	0.0	14.5	0.02	0.0001	-0.0019	-0.0018	4	4	8	12t*2	V	AS-D	Nala, Trees & Bushes	
67	290	250		AS	+0	0.4	0.0	14.5	0.39	0.0019	-0.0363	-0.0345	4	3	7	12t*2	V	AS-D	Bushes, Nala	
68	400	345		A	+1.5	1.0	0.0	24.0	10.53	0.0363	-0.0020	0.0343	7	6	13	12t*2	V	A-D	Bushes	Bushes
69	390	395		A	+1.5	0.8	0.0	24.0	0.79	0.0020	0.0064	0.0084	7	7	14	12t*2	V	A-D	Bushes, Rail Way	
70	360	375		A	+0	-1.0	0.0	22.5	-2.51	-0.0064	-0.0032	-0.0096	6	6	12	12t*2	V	A-D	Bushes, Nala	
71	350	355		A	+0	1.1	0.0	22.5	1.14	0.0032	-0.0066	-0.0035	6	5	11	12t*2	V	A-D	Sparco Building Wall	
72	350	350		A	+0	2.3	0.0	22.5	2.32	0.0066	-0.0075	-0.0008	6	5	11	12t*2	V	A-D		
73	310	330	R 17°37'	C	+0	2.9	0.0	22.2	2.61	0.0075	0.0082	0.0156	9	8	17	12t*2	==	C-D		
74																				

West Wharf P/S — b c / a d — Baldia G/S

LEDGER FOR INDIVIDUAL TOWER

Circuit Voltage : 220 kV
No. of Circuit : 2 cct.
Conductor : ACSR/AC 330 mm^2
Ground Wire : OPGW 190/90 mm^2

Tower No.	Span (m)	Wind Span (m)	Daviation Angle	Tower Type	Tower Extention	Ground Level Difference (m)	Formation Level (m)	Cond. Support Height (m)	Cond. Support Level Difference (m)	h1/S1	h2/S2	Σh/S	Catenary Ang. (°) For side	Catenary Ang. (°) Back side	Catenary Ang. (°) Total	Insulator Class	Insulator Type	Type of Foundation	Remarks Crossing	Remarks Land Item
74		310		A	-3	0.2	0.0	19.5	-2.53	-0.0082	-0.0023	-0.0104	5	5	10	12t*2	V	A-D		
	310																			
75		310		A	-3	0.7	0.0	19.5	0.70	0.0023	-0.0111	-0.0088	5	5	10	12t*2	V	A-D		
	310																		Poultry Forme	
76		350	R 27° 0'	C	+0	0.7	0.0	22.2	3.44	0.0111	-0.0006	0.0105	8	9	17	12t*2	= =	C-D		
	390																			
77		390		A	+0	-0.0	0.0	22.5	0.24	0.0006	0.0011	0.0017	7	7	14	12t*2	V	A-D		
	300																		Poultry Forme	
78		384		A	+0	-0.4	0.0	22.5	-0.44	-0.0011	-0.0032	-0.0044	7	6	13	12t*2	V	A-D		
	378																			
79		323		A	+0	1.2	0.0	22.5	1.22	0.0032	-0.0346	-0.0314	7	4	11	12t*2	V	A-D		
	267																		Cattle Farm	
80		215	L 27° 47'	C	+12	1.1	0.0	34.2	12.81	0.0346	-0.0081	0.0265	11	5	16	12t*2	= =	C-D		
	163																		Wall, 132kV Line, Tel Line Road, H.t Line	
81		299	R 3° 00'	A	+12	1.0	0.0	34.5	1.26	0.0081	0.0365	0.0446	3	9	12	12t*2	V	A-D		
	435																		Nala	
82		393		A	-3	0.4	0.0	19.5	-14.60	-0.0365	-0.0131	-0.0496	5	5	10	12t*3	V	A-D		
	350																		Nala	
83		347		A	+0	1.6	0.0	22.5	4.60	0.0131	0.0115	0.0246	7	6	13	12t*3	V	A-D		
	344																			
84		309	L 89° 3'	DR	+0	1.2	0.0	17.2	-3.96	-0.0115	-0.0051	-0.0166	8	7	15	12t*2	=|=	DR-D		
	273																			
85		332	R 90° 8'	DR	+0	1.4	0.0	17.2	1.36	0.0050	-0.0517	-0.0467	7	6	13	12t*3	=|=	DR-D		
	390																		Road, 132kV Line	Urban Area
86		400		A	+12	2.9	0.0	34.5	30.17	0.0517	0.0045	0.0562	10	7	17	12t*2	V	A-D		
	410																		Road	
87		400		A	+4.5	5.7	0.0	27.0	-1.83	-0.0045	-0.0106	-0.0151	7	6	13	12t*2	V	A-D		
	390																		Road	
88		357		A	+3	5.6	0.0	25.5	4.13	0.0106	0.0072	0.0178	7	7	14	12t*2	V	A-D		
	323																		Road	
89		262	R 89° 4'	DR	+0	5.6	0.0	17.2	-2.68	-0.0072	0.0110	0.0037	8	7	15	12t*2	=|=	DR-D		
	200																		Nala	
90		200		AS	+0	0.5	0.0	14.5	-2.19	-0.0110	0.0066	-0.0043	3	4	7	12t*2	V	AS-D		
	199																			
91		155	R 90° 0'	DR	-6	1.9	0.0	11.2	-1.32	-0.0066	0.0357	0.0291	5	6	11	12t*2	=|=	DR-D		
	110																		Boundary Wall, Road, Nala	
92		70		D	-3	-6.9	0.0	14.2	-3.93	-0.0357	0.1033	0.1676	2	21	23	12t*2 12t*1	= −	D-D		
	30																			
G3				Gantry	+0	-0.9	0.0	12.0	-3.10							12t*1	−|	MONO BLOCK FOUNDATION		

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