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DESIGN OF PILE FOUNDATION SUBJECTED TO LATERAL LOAD

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水平荷重を受けるくい基礎の設計

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1. ま え が き

日本は地震国であるから、構造物の設計にさいし、この影響を考慮する必要がある。このことは道路橋示方書（建設省道路局長通達）第1編共通編1.8.1「荷重の種類」の項に規定されており、設計震度等の具体的事項については、第5編耐震設計指針に規定されている。

道路橋下部構造の設計は、第4編下部構造設計指針によって行なわれている。この指針の中のくい基礎の設計に関する規定は、1964年に制定された古いものであるから、その後の技術の進歩、データの集積に応じて現在改訂作業を行なっているところである。

本論文では、改訂のために検討されたいくつかの事項のうち、主として地震荷重のような水平荷重を受けるくい基礎の設計に関する事項を選んで、設計の基本的な考え方と方法を論じることとするものである。これらの事項は、まだ検討中の段階であるが、重要な部分は「道路橋示方書」の条文に取入れられるものと思われる。したがって本論文の内容は、日本の道路橋設計の一つの大綱を示すものである。

なお本論文の構成は次のとおりであり、それぞれの章の概略の内容と意義を述べる。

2. 「くい本体の設計」では、くい頭反力と変位の算出方法に関し、3つの力の釣合いのみから解く慣用法と、変位を考慮した厳密解とを紹介した。通常の基礎においては簡単な慣用法で十分であるが、これが十分適用できず誤差が大きくなる場合があるので、慣用法の適用限界を示し、この場合には厳密解によることを示唆した。一般にはくいは十分長いと考えてよいが、短かい場合には上の方法を一部修正して適用する必要があるので、その取扱いを示した。

3. 「横方向地盤反力係数の算定」では、弾性床土上の梁の理論を応用し、くい応力と変位の解析法に用いる横方向地盤反力係数を、土質試験結果から推定する方法を述べたものである。この方法は、建設省土木研究所において約150例の水平載荷試験の結果を解析し、各種の土質試験との間の相関を求める実験とに基づいて提案されたものである。

4. 「くい頭部の設計」は、くい頭部とフーチングの結合の設計法を述べたものである。くい頭部のフーチングの結合の設計法には具体的な方法がなかったが、建設省土木研究所では、結合部



の剛性と強度を確実なものとするため設計基準を作成し、基準の妥当性を検証するため実物大に近い規模の載荷試験を行なった。この章では、結合部の設計方法の基本的考え方と実験結果とを述べる。

5. 「自由長を有するくい基礎の挙動」においては、自由長を有するくい基礎の水平抵抗と振動性状実験の結果を述べたものである。フーチングが地中に埋っている一般のくい基礎では、振動実験等の実績も多く、また耐震性についても実地震により実証済みであるが、自由長を有するくい基礎については、新形式の基礎であるためほとんど実験が行なわれていなかった。本章では、建設省土木研究所で行なった実物大に近い規模の実験の結果を述べたものである。

2. くい本体の設計

2-1 くい頭反力および変位の算定方法

図-1に示すようなくい頭が剛結されているくい基礎において、フーチングに作用する外力 (V_0, H_0, M_0) に対して各くいの頭部に生じる反力および変位の計算法には、厳密法と慣用法とがある。ただし、いずれの場合にもフーチングは剛体とし地盤はくい軸方向およびくい軸直角方向の弾性的なバネとして扱っている。

なお、ここでは、くいが十分長く無限長とみなせる場合の計算法を示す。この計算法の適用範囲および短かくいの計算法については2-2「短かくいの設計」で述べる。

(1) 厳密法

フーチングの水平変位を δ_x , 鉛直変位を δ_y , 回転角を α とするとこれらは次の三元連立方程式を解いて求められる。

$$\begin{cases} A_{xx} \delta_x + A_{xy} \delta_y + A_{x\alpha} \alpha = H_0 \\ A_{yx} \delta_x + A_{yy} \delta_y + A_{y\alpha} \alpha = V_0 \\ A_{\alpha x} \delta_x + A_{\alpha y} \delta_y + A_{\alpha\alpha} \alpha = M_0 \end{cases} \quad (1)$$

ここで

$$\begin{aligned} A_{xx} &= \sum_i (K_1 \cos^2 \theta_i + K_v \sin^2 \theta_i) \\ A_{xy} &= A_{yx} = \sum_i (K_v - K_1) \sin \theta_i \cos \theta_i \\ A_{x\alpha} &= A_{\alpha x} = \sum_i \{ (K_v - K_1) x_i \sin \theta_i \cos \theta_i - K_2 \cos \theta_i \} \\ A_{yy} &= \sum_i (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i) \\ A_{y\alpha} &= A_{\alpha y} = \sum_i \{ (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i) x_i + K_2 \sin \theta_i \} \end{aligned}$$

$$A\alpha = \sum_i \{ (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i) x_i^2 + (K_2 + K_3) x_i \sin \theta_i + K_4$$

$$K_1 = \frac{12EI\beta^3}{(1+\beta h)^3 + 2}$$

$$K_2 = K_3 = K_1 \frac{1 + \beta h}{2\beta}$$

$$K_4 = \frac{4EI\beta(1+\beta h)^3 + 0.5}{1 + \beta h(1+\beta h)^3 + 2}$$

$$\beta = \sqrt[4]{\frac{K_H D}{4EI}}$$

D : くい径 (cm)

EI : くの曲げ剛性 (Kg・cm²)

K_H : くい軸直角方向地盤反力係数 (Kg/cm²)

K_v : くい軸方向バネ定数 (Kg/cm)

(1) 式から δ_x , δ_y , α が求まると, 各くいの頭部に作用するくい軸方向力 P_{Ni} , くい軸直角方向力 P_{Hi} , くい頭モーメント M_i は次式により計算できる。

$$\begin{cases} P_{Ni} = K_v \{ \delta_x \sin \theta_i + (\delta_y + \alpha x_i) \cos \theta_i \\ P_{Hi} = K_1 \{ \delta_x \cos \theta_i - (\delta_y + \alpha x_i) \sin \theta_i \} - K_2 \alpha \\ M_i = -K_3 \{ \delta_x \cos \theta_i - (\delta_y + \alpha x_i) \sin \theta_i \} + K_4 \alpha \end{cases} \quad (2)$$

(2) 慣用法

設計上の便宜を考えて, ある適用範囲のもとで次式による近似計算が可能である。

(鉛直くいのみの場合)

$$\begin{cases} P_{Ni} = \frac{V_0}{n} + \frac{M_0 + \lambda H_0}{\sum_i x_i^2} x_i \\ P_{Hi} = \frac{H_0}{n} \\ M_i = -\lambda P_{Hi} \end{cases} \quad (3)$$

$$\left\{ \begin{array}{l} \delta_x = \frac{H_0}{n K_1} \\ \delta_y = \frac{V_0}{n K_v} \\ \alpha = \frac{M_0 + \lambda H_0}{K_v \sum_i x_i^2} \end{array} \right.$$

ここで

$$\lambda = \frac{1}{2} \left(h + \frac{1}{\rho} \right)$$

(斜ぐいを含む場合)

図-1を参照して

$$\left\{ \begin{array}{l} V_i = \frac{V_0}{n} + \frac{M_0 + \lambda H_0}{\sum x_i^2} x_i \\ H_i = V_i \tan \theta_i + \frac{\sec \theta_i}{\sum_i \sec \theta_i} (H_0 - \sum_i V_i \tan \theta_i) \\ P_{N_i} = V_i \cos \theta_i + H_i \sin \theta_i \\ P_{H_i} = -V_i \sin \theta_i + H_i \cos \theta_i \\ M_i = -\lambda P_{H_i} \\ \delta_x = \frac{H_0}{\sum_i (K_1 \cos^2 \theta_i + K_v \sin^2 \theta_i)} \\ \delta_y = \frac{V_0}{n K_v} \\ \alpha = \frac{M_0 + \lambda H_0}{K_v \sum_i x_i^2} \end{array} \right. \quad (4)$$

慣用法の適用限界

くい基礎が対称配置の鉛直ぐいのみで構成されている場合には, $K_v = \infty$ であれば, くい頭反

力および水平変位について厳密法と慣用法は完全に一致する。一方、 $K_v=0$ であれば各くいの挙動は単ぐいのそれと同じになる。実際のくい基礎ではこの両者の中間の状態にあり、次式で定義される無次元パラメーター ϕ によって鉛直方向と水平方向の剛性比が表現される。

$$\phi = \frac{K_v \sum_i x_i^2}{K_v \sum_i x_i^2 + \frac{nEI\beta}{1+\beta h}} \quad [0 \leq \phi \leq 1] \quad (5)$$

ここで n : くい本数

このパラメーター ϕ を使えば、くい頭反力、くい頭変位とも、厳密法による解(f とする)は次式のように簡潔に表現できる。

$$f = f_{(K_v=0)} + \phi \{ f_{(K_v=\infty)} - f_{(K_v=0)} \}$$

ここで $f_{(K_v=0)}$ は $K_v=0$ とした時の解で単ぐいの計算値になり、 $f_{(K_v=\infty)}$ は $K_v=\infty$ とした時の解で慣用法による計算値になる。

すなわち、くい基礎全体として水平方向の剛性に比べて鉛直方向の剛性が大きいほどパラメーター ϕ は1に近くなり、従って厳密法と慣用法の差が小さくなる。各種の基礎について試算した結果によると $\phi > 0.95$ であれば両計算法による差はほぼ2割以下になることが確かめられた。

斜ぐいを含む場合には、上記のような簡単なパラメーターによる適用限界の判定はできないが、傾向的には同じようなことがいえ、鉛直方向の剛性が小さい時、斜角がかなり大きい時などには慣用法による誤差が増大する。

2-2 短いくいの設計

一般の設計でおこなわれているようにくいを弾性床上の梁とみなすと、図-2のモデルにおいてくいの変位 y は次の4階の微分方程式で表わされる

$$EI \frac{d^4 y}{dx^4} + K_H D y = 0 \quad (6)$$

境界条件を入れて解くと、くい頭に水平力 H_0 が作用した時のくい頭変位量 δ は

$$\delta = \frac{1}{\alpha} \cdot \frac{2\beta H_0}{K_H D} \quad (7)$$

で表わされる。ここで $\frac{2\beta H_0}{K_H D}$ は ℓ (くい長) が無限長の場合のくい頭変位量であり、係

数 α はくいが有限長であることによる補正係数で $\beta \ell$ のみの関数で表わされる。一方、くいを剛体とみなした時のくい頭変位量は $\delta = \frac{4H_0}{K_H D \ell}$ となるので(7)式の係数 α は $\alpha = \frac{\beta \ell}{2}$ となる。

図-3 は α と $\beta \ell$ の関係を表示したものであり、 $\beta \ell > 3$ であれば $\alpha \div 1$ で無限長の解に近くなり、 $\beta \ell < 1$ であれば $\alpha \div \frac{\beta \ell}{2}$ で剛体の解に近くなることがわかる。

上記は水平力が作用した時のくい頭変位のみに着目したが、他の場合でも同様の傾向にあり、結局、地盤中の基礎の水平挙動に関しては設計理念上次のような分類が可能である。

$\beta \ell \geq 3$ 弾性体，無限長

$1 < \beta \ell < 3$ 弾性体，有限長

$\beta \ell \leq 1$ 剛体

“剛体”領域と“弾性体・無限長”領域の間にあるくいは一般に“短いくい”と呼ばれており、くい先端の境界条件を計算に考慮しなければならない。

図-4 は $\phi 2m$ 、 $\ell 17m$ の PC ウェルについて水平載荷試験を行った概要であるが、この場合 $\beta \ell \div 2$ で“短いくい”の領域に入り、先端は砂礫層に約 5m 根入れされている。図-5 は測定した傾斜角の分布と、先端の境界条件を free, hinged, fixed の3種に変えて計算した値とを比較したものである。この場合のように先端がかなり砂礫層に根入れされている時には、先端を hinged と仮定すると計算値と実測値はよく適合する。

図-6 はくい先端支持条件がくい体のたわみ分布、曲げモーメント分布に与える影響を図示したものである。

3. 横方向地盤反力係数 (K_H)

くいの水平抵抗を支配する最大の要素は、地盤の水平方向の抵抗特性である。地盤の荷重-変形特性は一般に非線形である上に、くいのようなフレキシブルな物体を介して載荷された場合には、上から順次逐次破壊を起こしていくので、明確な降伏点は定めがたい。そこで設計においては、地盤調査の精度も考慮して地盤を弾性的なバネに置きかえ、くいをこの弾性床上に支持された梁とみなして計算をし、許容変位量とくい体の許容応力度から設計荷重を決めている。この場合、地盤は当然非線形であるので、基準となる地表面変位量を定め、それに対する見かけのばね定数を用いることになる。これが横方向地盤反力係数 K_H であり、この推定方法を見出すために

いくつかの実験を行なった。

まず、横方向地盤反力係数におよぼす載荷幅の影響を調べるために、砂およびローム地盤内にテストピットを掘削し、径(D)が30, 60, 90, 120 cmの円形載荷板を使って水平方向平板載荷試験を行なった。D = 30 cmの時の K_H を K_{30} とし、他の径に対する値を K_{30} との比で表示したのが図-7であり、地盤深さおよび深度にほとんどかわりなく、 K_H は載荷幅の $\frac{3}{4}$ 乗に逆比例して減少することがわかった。すなわち

$$\frac{K_H}{K_{30}} = \left(\frac{D}{30} \right)^{-\frac{3}{4}} \quad (8)$$

となる。

今、30 cmの載荷板による平板載荷試験結果から求めた見かけの変形係数を E_{30} とし、これを地盤の特性値と考える。一方、各種の地盤調査法における相互の関連を調べるためにボーリング孔円載荷試験から求めた変形係数 E_P 、サンプリング試料による一軸または三軸試験から求めた変形係数 E_c 、標準貫入試験値 N と E_{30} との比較実験を行ない、図-8, 図-9に示すような結果から近似的に次に示す実験式を得た。

$$E_{30} = 4 E_P = 4 E_c = 28 N \quad (9)$$

(8)と(9)の関係式に、くいの載荷試験結果から実験的に定めた比例定数を付加して、くいの横方向地盤反力係数 K_H の推定式として次式が提案される。

$$K_H = \frac{E_{30}}{4.8 D^{\frac{3}{4}}} = \frac{E_P}{1.2 D^{\frac{3}{4}}} = \frac{E_c}{1.2 D^{\frac{3}{4}}} = \frac{7 N}{1.2 D^{\frac{3}{4}}} \quad (10)$$

図-10は、ローム地盤で行なった単ぐいの載荷試験結果と(8)式との対比を示すものであり、図-11は全国的に150例の実ぐいの水平載荷試験例を集め、(10)式に基づいて $7N/K_H$ とくいの径(D)の関係を図示した。ここで、 K_H としては地表面変位量が10 mmに対する値を用い、標準貫入試験

値 N としては、地表面から $1/\beta$ $\left(\beta = \sqrt[4]{\frac{K_H D}{4 E I}} \right)$ 程度の深さまでの平均値を使った。

4. くいの頭部の設計

上部構造あるいは下部構造の躯体部に作用する荷重は確実に基礎構造部に伝達されることが必要であるから、基礎構造物と躯体部との結合部の検討は重要である。特にくい基礎の場合には断面が急変する個所になるので、円滑に荷重が伝達されなければならない。

現在、道路橋の下部構造指針では、くいとフーチングの結合部は原則として剛結として設計するよ

うに定められ、次の2つの接合方法のうちいずれかによるとされている。

方法A：フーチングの中にくいを一定長さだけ埋込み、埋込んだ部分によってくい頭拘束モーメントに抵抗する方法。

鋼管くい、PCくい、RCくいにはこの方法が通用でき図-12で(1)、(2)に示す結合方法がこれに属する。

方法B：フーチング内へのくいの埋込み長さは小さいが主として鉄筋で補強してそれによりくい頭拘束モーメントに抵抗する方法。

鋼管くい、PCくい、RCくい、場所打ち鉄筋コンクリートくいのいずれにも適用することができ図-12で(3)～(5)に示す方法がこれに属する。

設計法は、くい種類および方法A、Bごとに定められているが、1例として方法Bで結合された鋼管くいの設計法を紹介する。

- (1) くいとフーチングの結合は剛結として設計し、くい頭部に作用する押込み力ないしは引抜き力、水平力およびモーメントの全てに対して抵抗できるように設計する。
- (2) フーチングへのくい頭部の埋込みは10cmとする。
- (3) フーチングコンクリートの垂直支圧応力度（図-13参照）

$$\sigma_{cv} = \frac{P}{\frac{\pi}{4} D^2} \leq \sigma_{ca}$$

- (4) フーチングコンクリートの水平支圧応力度

$$\sigma_{ch} = \frac{H}{D \ell} \leq \sigma_{ca}$$

- (5) フーチングコンクリートの押し抜きせん断応力度

$$\tau = \frac{P}{\pi (D + h_1) h_1} \leq \tau_a$$

- (6) 蓋板と十字補強板

$$t_1 = t_2 = 1.2 \text{ cm} \quad h_2 = 30 \text{ cm}$$

ただし、くい径1m以内に適用する。

t_1 ：蓋板の板厚（cm）

t_2 ：十字補強板の板厚（cm）

h_2 ：十字補強板の高さ（cm）

(7) 仮想鉄筋コンクリート断面の応力度

押込力 P とモーメント M , または引抜き力 P_t とモーメント M が作用する場合には, 鉄筋コンクリート断面を仮定して, コンクリートと鉄筋の応力度を検討する。

(8) 鉄筋の定着

(I) 溶接部のせん断応力度

$$\tau_s = \frac{\sigma_{sa} A_{st}}{1.4 \lambda \ell_0} \leq \tau_{sa}$$

σ_{sa} : 鉄筋の許容引張応力度 (Kg/cm^2)

A_{st} : 鉄筋の断面積 (cm^2)

λ : すみ肉溶接の脚長 (cm)

ℓ_0 : 溶接長 (cm)

(II) 鉄筋の定着長

$$L_0 = \frac{\tau_{sa} A_{st}}{\tau_{0a} \cdot u}$$

τ_{0a} : コンクリートの許容付着応力度 (Kg/cm^2)

u : 鉄筋の周長 (cm)

(9) フーチング端部のくいについては水平方向の押抜きせん断応力度について検討する。

次に, 方法 B で結合された鋼管くいに関する模型実験の結果を述べる。図-14 は模型の形状寸法である。

図-15 に示すように, くい頭結合部は 10 cm の埋込みを有し, 16 mm の鉄筋 8 本で補強されている。くい頭部には 16 mm 厚の鋼板の蓋が取り付けられてあり, 鋼管くいの周りには円周方向の引張応力に抵抗できるように 16 mm のリング筋が入れている。

鉛直荷重は最大 90 t 載荷し, その時のフーチングコンクリート, くい体, 蓋板の応力分布を図-16 に, 実測応力度と計算値との対比を図-17 に示す。ただし, 蓋板応力の計算値は全支圧応力が一様に蓋板に作用すると仮定して

$$\sigma = \frac{99}{320} \sigma_c \left(\frac{D}{t} \right)^2$$

σ_c : 支圧応力度 (Kg/cm^2)

D : くい径 (cm)

t : くいの肉厚 (cm)

によって求めたが、実際にはくい体への鉛直荷重の伝達はいく肉厚部に集中しており、蓋板の実測応力は計算値よりかなり小さい。

次に鉛直荷重を除荷し水平荷重を最大70 t まで載荷した。 P_1 、 P_4 くいについて図18に実測した応力値と計算値の比較を示す。

この結果より、埋込み側のくい P_1 と引抜き側のくい P_4 とでは明らかにその挙動に相違がある。支圧応力度の分布より P_1 では全面が圧縮されているが、 P_4 ではくい埋込み部で回転を生じている。このことは明らかに P_4 の拘束が弱いことを意味し、くい頭を完全固定状態にすることは著しく困難であることがわかる。

しかしながら全体の挙動としては、現在定められているくい頭結合方法は設計上十分な耐力を有しており、フーチング内部に発生する応力は一般に計算値より安全側の値になっている。

5. 自由長を有するくい基礎の挙動

大口径の柱群を所定の支持層まで根入れさせ、頭部を頂版により剛結した形式の多柱基礎は、水中における施工性にすぐれるため最近施工例がふえ、本州四国連絡橋のような大型基礎の計画にも含まれている。こうした自由長をもつくい群の水平抵抗特性と振動性状を調べる目的で行なった大型の模型実験の結果を以下に述べる。

図-19に示すような頂版厚さを異にする2基の模型を作り、水平載荷試験、自由振動試験、強制振動試験(頂版上)を行なった。記号A、B、Cは載荷位置と載荷方向を示すものである。

水平載荷試験から平均の横方向地盤反力係数(K_H)と地表面変位の関係を図示したのが図-20であり、群くい効果によって単ぐいの場合より K_H は低減し、また載荷位置によっても変化する。図-21は各くいごとの K_H の相異を中央のくいを基準とした比率で表示したもので、前列のくいの分担比が他のくいに比べて5割程度大きい。図-22は偏心荷重をかけた場合(載荷位置B)のねじりばね定数を表示したものである。この場合、ねじり抵抗のうちくい躯体のねじり抵抗によるものが80%で地盤の水平反力によるものは20%程度の割合であり、基礎の剛性の低い多柱形式では“ねじり”に対するチェックが必要となる。

図-23はタワー天端位置(C)自由振動による加速度モードであり、表-1、表-2は自由振動試験および強制振動試験から求めた固有振動数と減衰定数の結果である。多柱基礎の固有周期を1自由度系のモデルに置きかえて求めるものとし、振動に参与する質量として地表面上の全質量をとり、測定された固有周期より逆に地盤の K_H を計算すると

$$\text{頂版軸心自由振動 (A)} \rightarrow K = 1.23 \text{ kg/cm}^3$$

$$\text{タワー軸心自由振動 (C)} \rightarrow K = 1.24 \text{ kg/cm}^3$$

$$\text{強制振動 (頂版位置)} \rightarrow K = 1.08 \text{ kg/cm}^3$$

となり、静的載荷による K_H と比べて同程度かやや低い値となっている。

あ　と　が　き

本論文では水平荷重を受けるくい基礎の設計の基本的事項をいくつか取上げて，日本の道路橋設計における現況を述べた。今後，水平荷重を受けるくい基礎の設計に関し，基準化のために検討を要すべき事項としては，群ぐいの変形機構の問題があり，現在研究を実施中である。

くい頭部の設計については，本論文では補強鉄筋による方法のみ紹介したが，埋込み式の実験もあわせて行なっているので後日報告することとしたい。

表一 1 固有振動数 f (%)、減衰定数 h (自由振動)

加 速 度 計 設 置 位 置		頂版 軸心自由		頂版 偏心自由		タワ- 軸心自由	
		f	h	f	h	f	h
No. I	頂版上中央軸心方向	85	0.074	85	0.088	78	0.068
	// 軸心直角方向	—	—	95	0.058	—	—
	頂版上隅角部軸心方向	85	0.071	80	0.058	78	0.062
	// 軸心直角方向	—	—	152	0.064	—	—
	中央杭 地上 2 m 軸心方向	85	0.075	83	0.086	78	0.075
	// 地上 0.3m //	85	0.088	83	0.097	78	0.071
No. II	頂版上中央軸心方向	95	0.128	100	0.153	83	0.058
II	// 軸心直角方向	—	—	130	0.077	—	—

表一 2 共振振動数

f' (%)
 減衰常数 h
 固有振動数
 f (%)
 $f = f' / \sqrt{1 - 2h^2}$
 (No. I 中央杭
 地上 1.7 m 地点
 の歪測定より)

起振力(t)	f' (%)	h	f
0.3	78	0.073	785
0.3	785	0.078	79
0.6	785	0.078	79
0.6	785	0.080	79
1.3	765	0.087	77
1.8	750	0.066	755

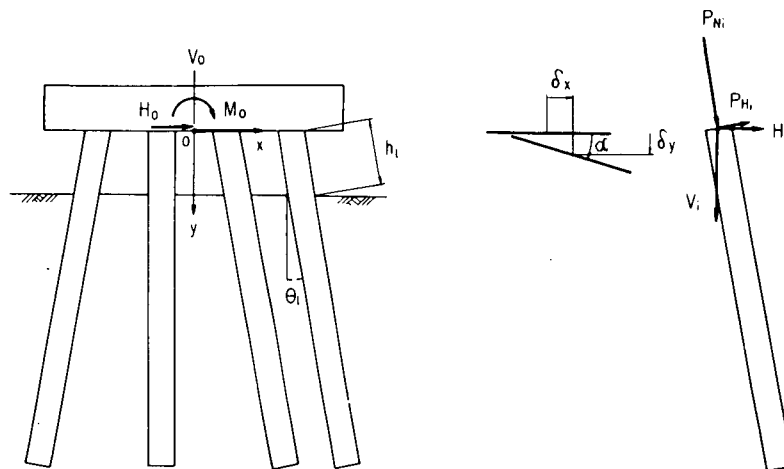


図-1 くい基礎の計算モデル

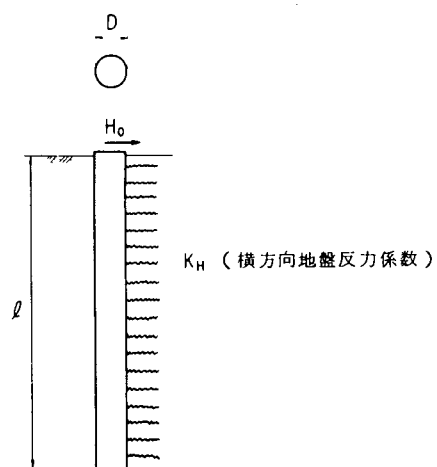


図-2 単ぐいの計算モデル

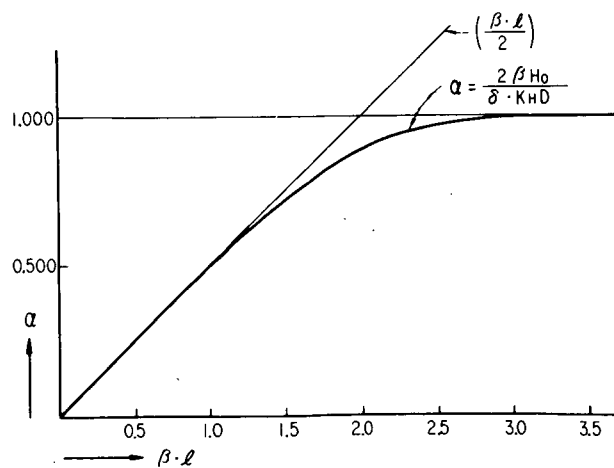


図-3 $\alpha - \beta l$ 図

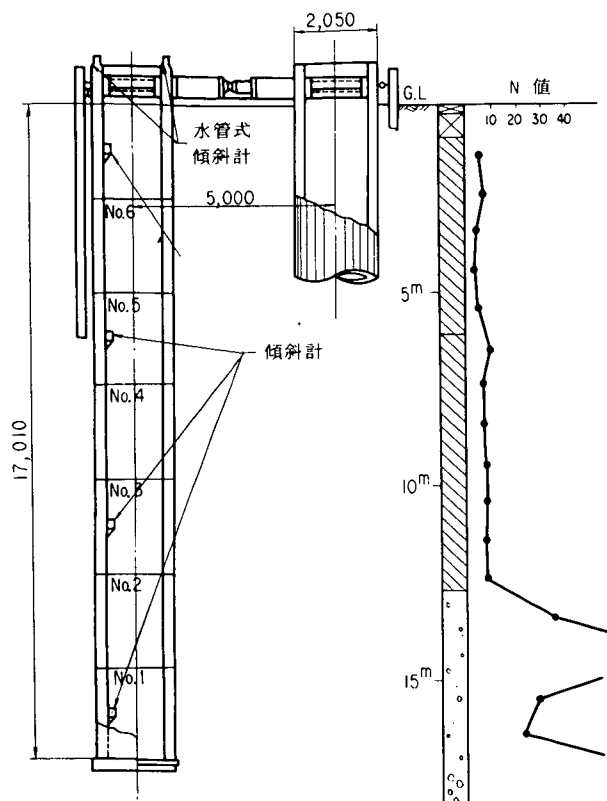


図-4 PCウエル水平載荷試験概要
および地盤柱状図

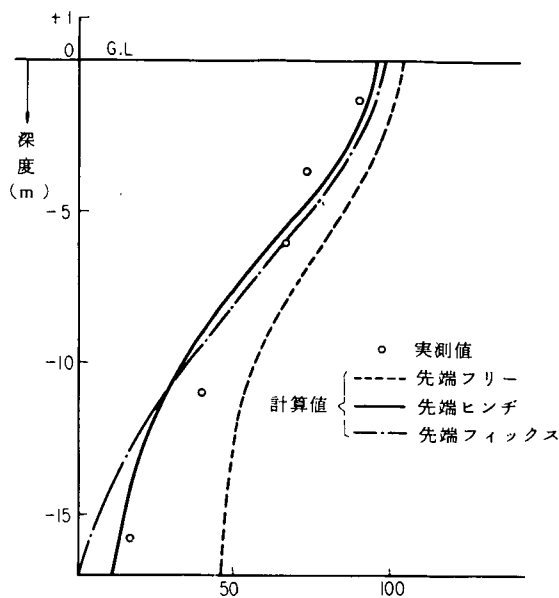


図-5 傾斜角分布図
(計算値との比較) 荷重 50 t

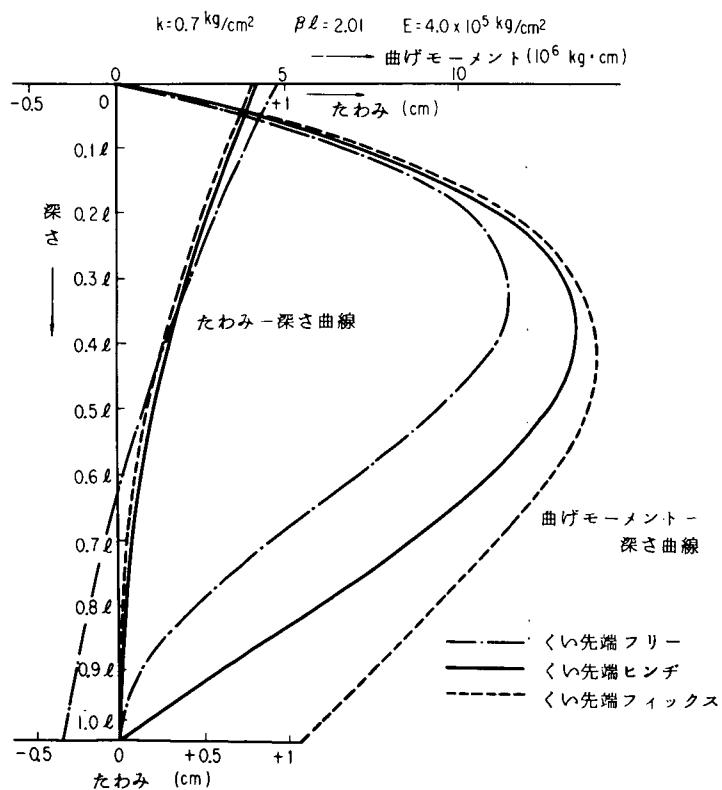


図-6 くい先端支持条件による相違

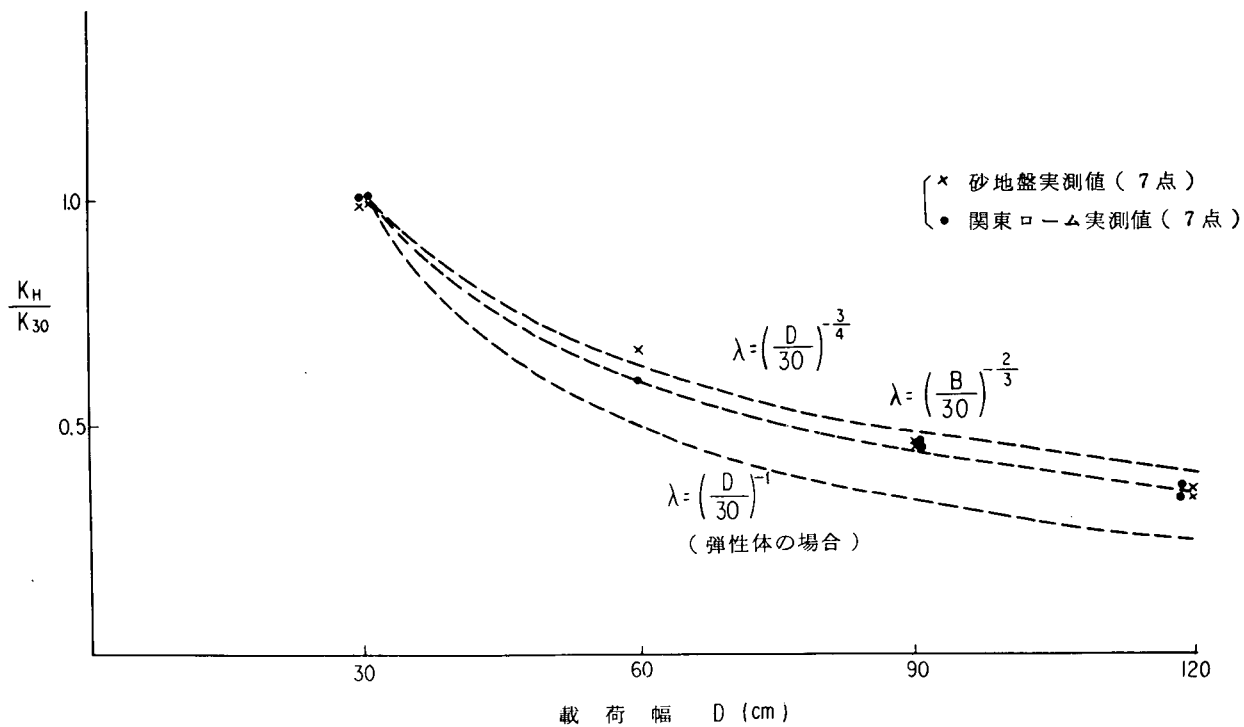


図-7 載荷幅と地盤反力係数 (K 値)

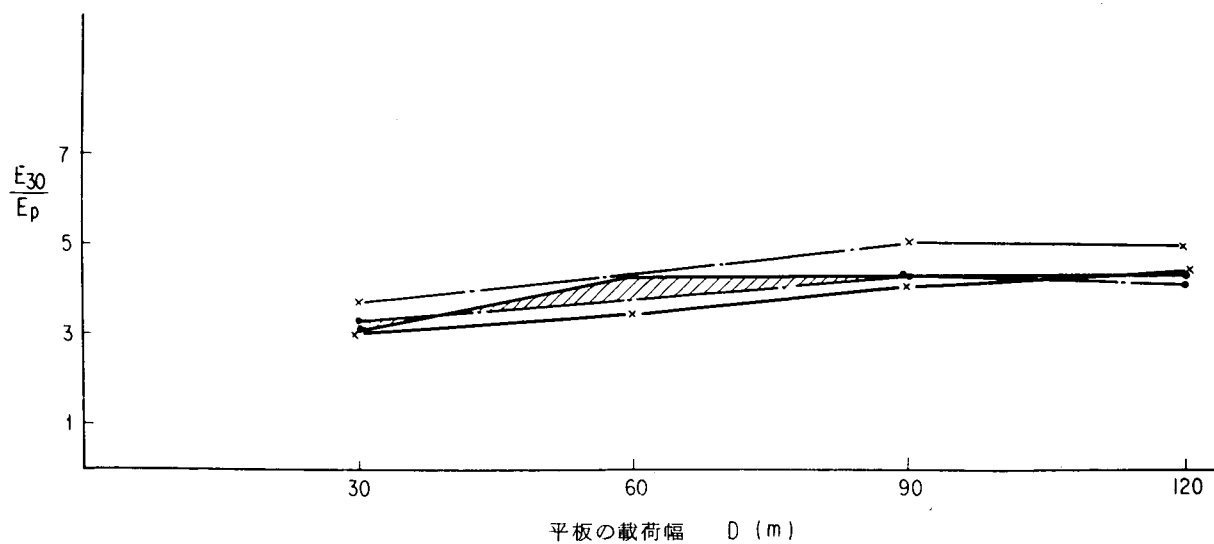


図-8 ボーリング孔内変形係数 E_p と
水平方向平板載荷による E_{30} との比較

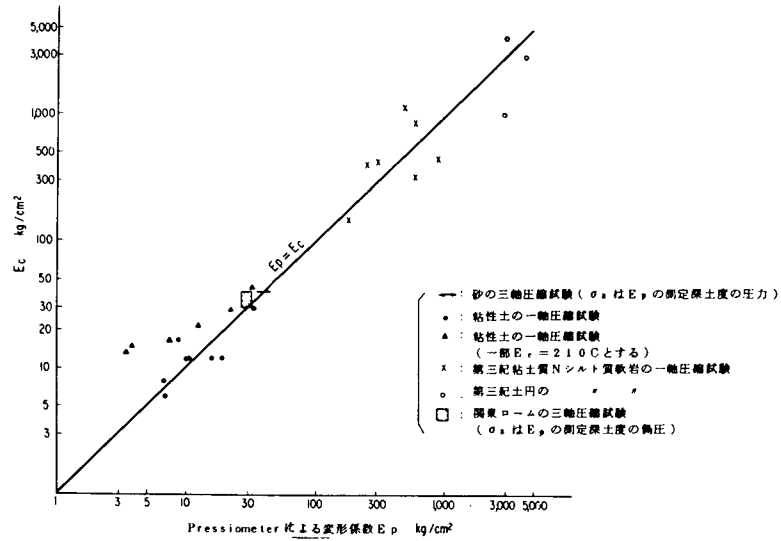


図-9 室内土質試験より求めた変形係数とプレシオメータ変形係数の関係

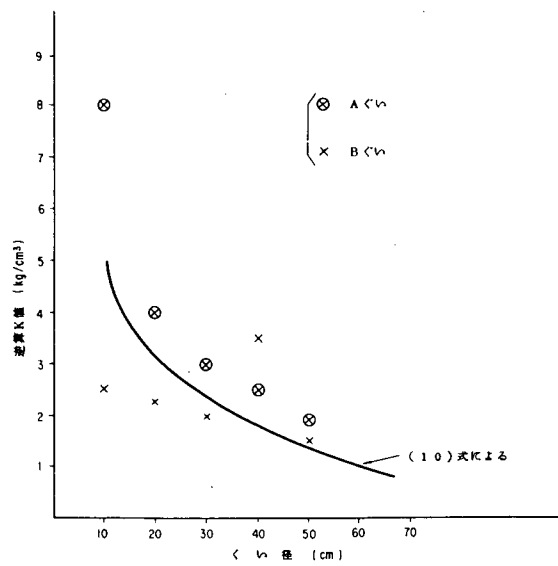


図-10 関東ローム地盤における単ぐいの水平載荷試験結果

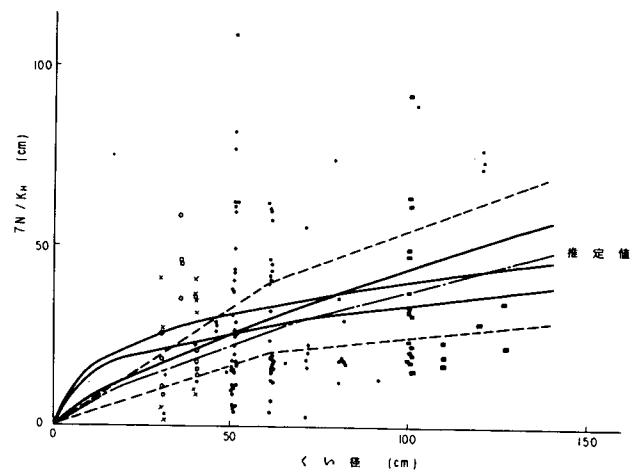
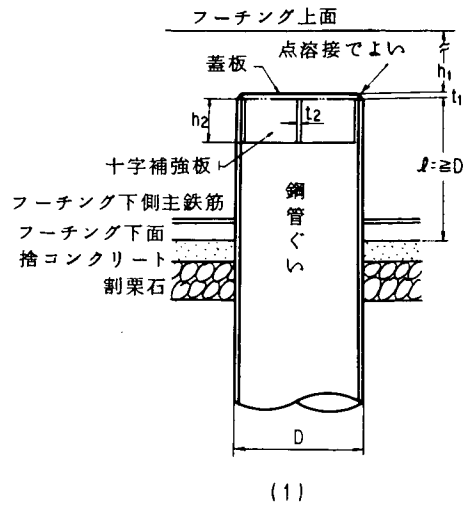
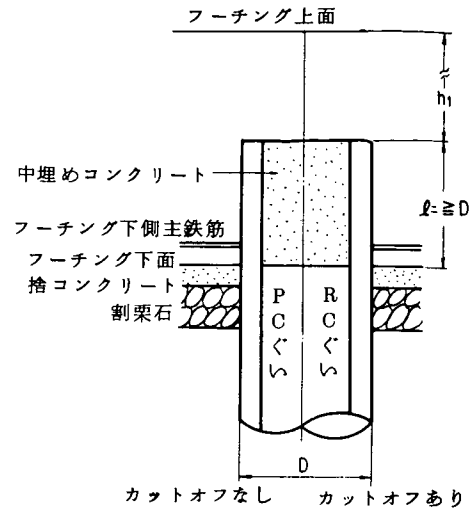


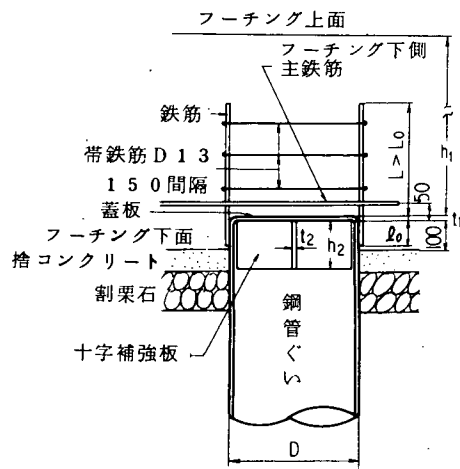
図-11 くい径と K_H



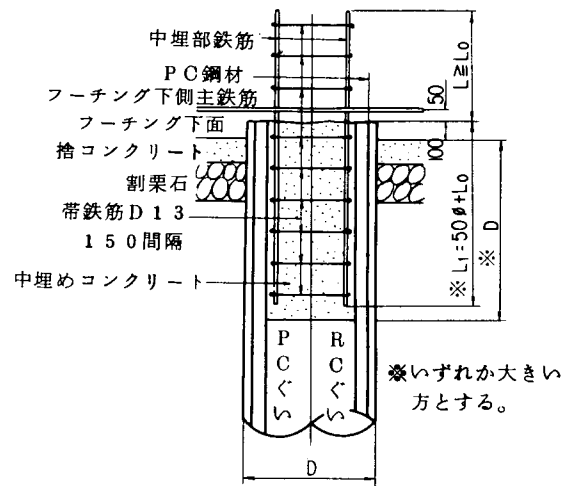
(1)



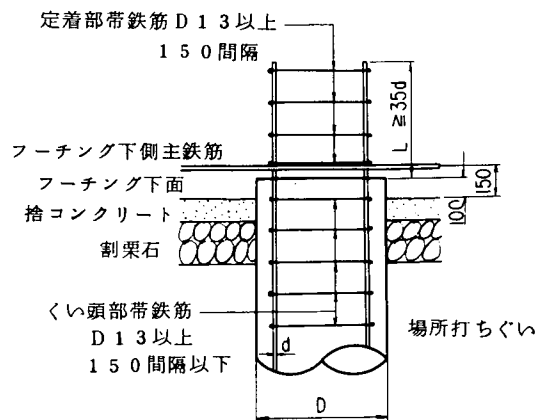
(2)



(3)

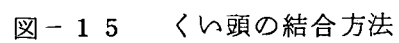
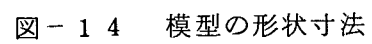
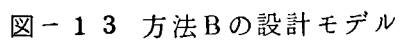


(4)



(5)

図-12 くい頭結合方法



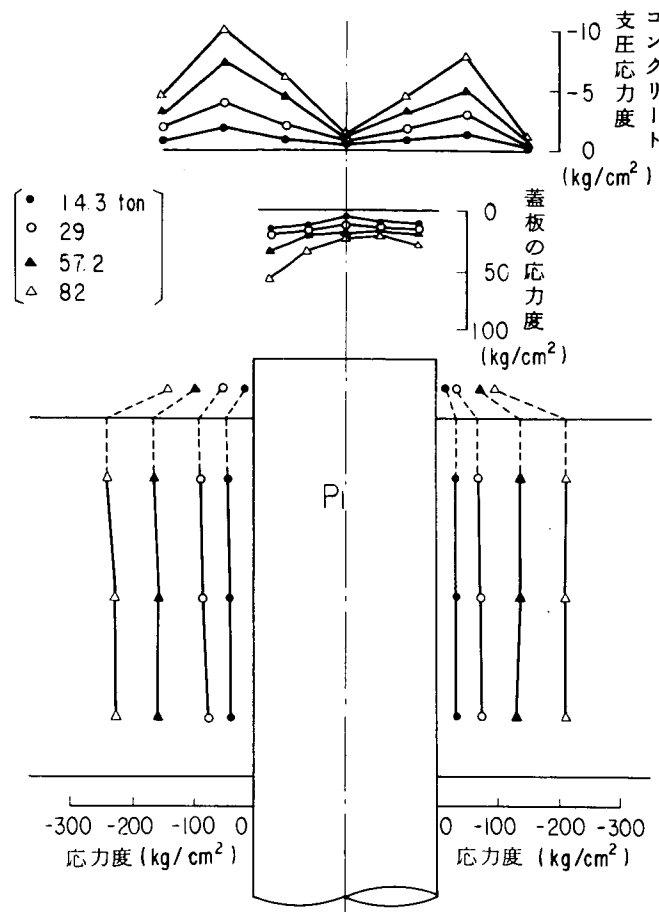


図-16 応力分布 (p_1)

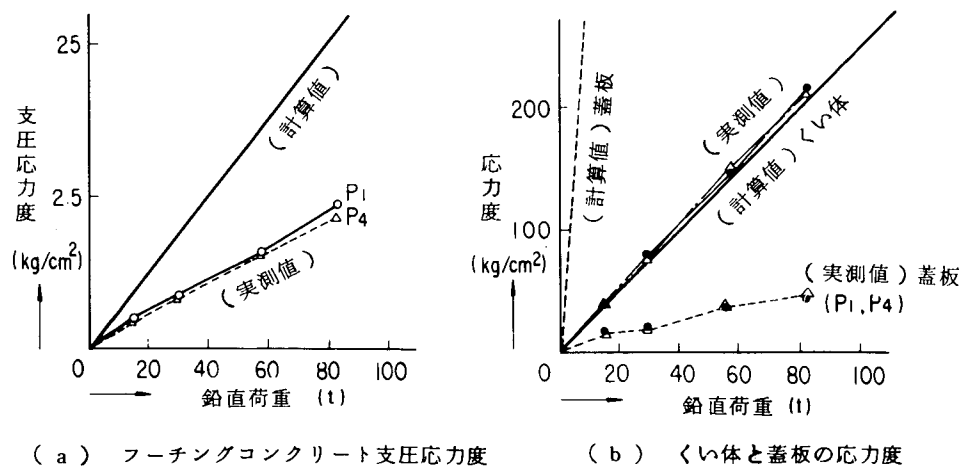


図-17 計算値の実測値の比較

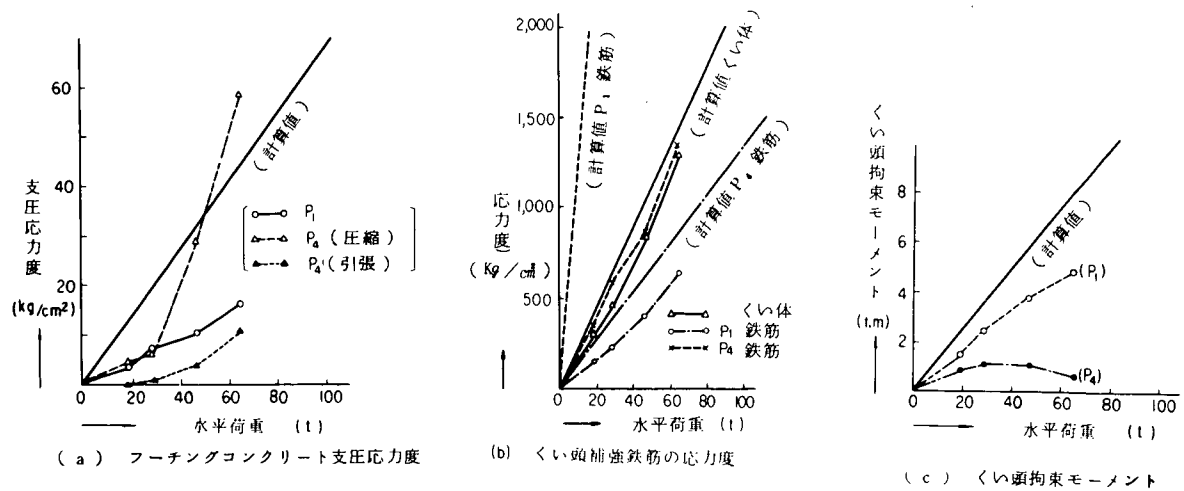


図-18 計算値と実測値との比較

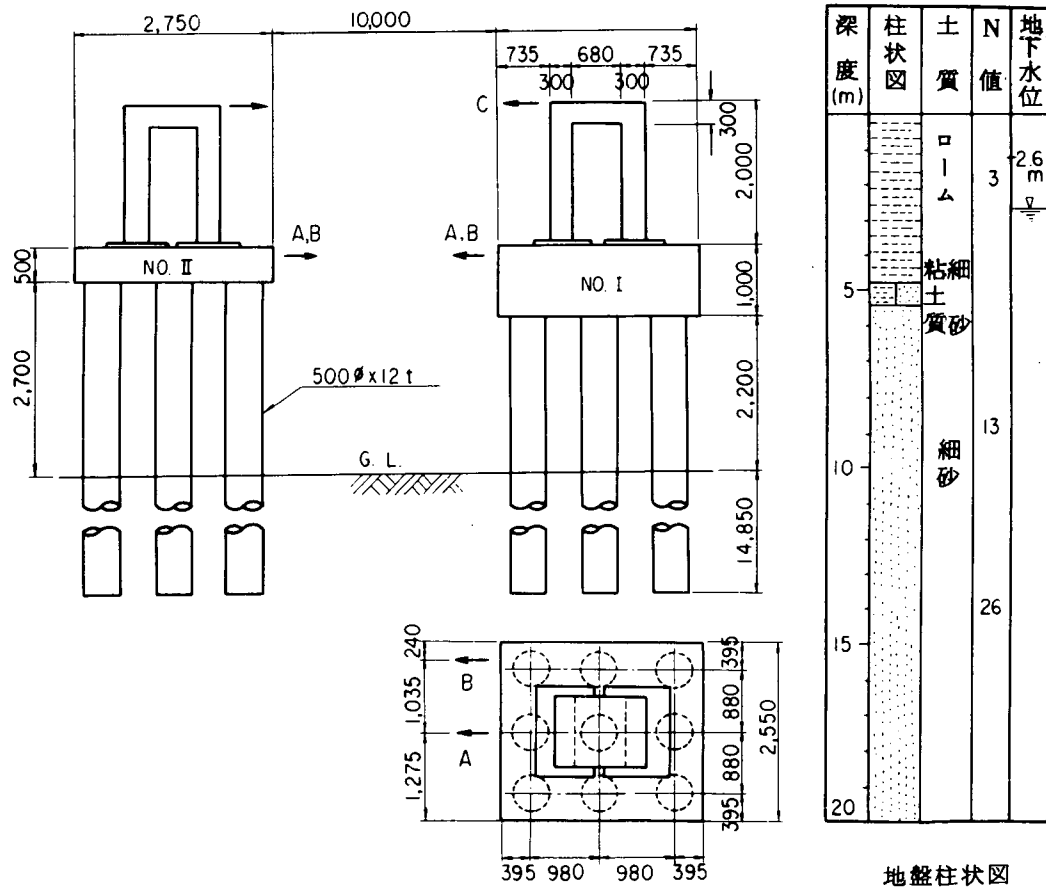


図-19 模型の形状・寸法

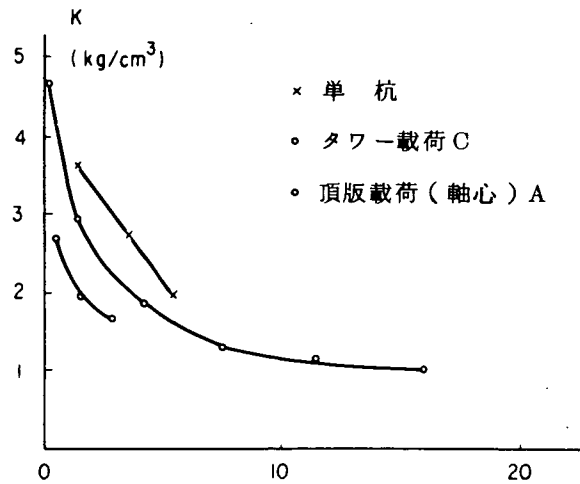


図 - 20 地表面変位 - 横方向地盤反力係数 K_H (平均)

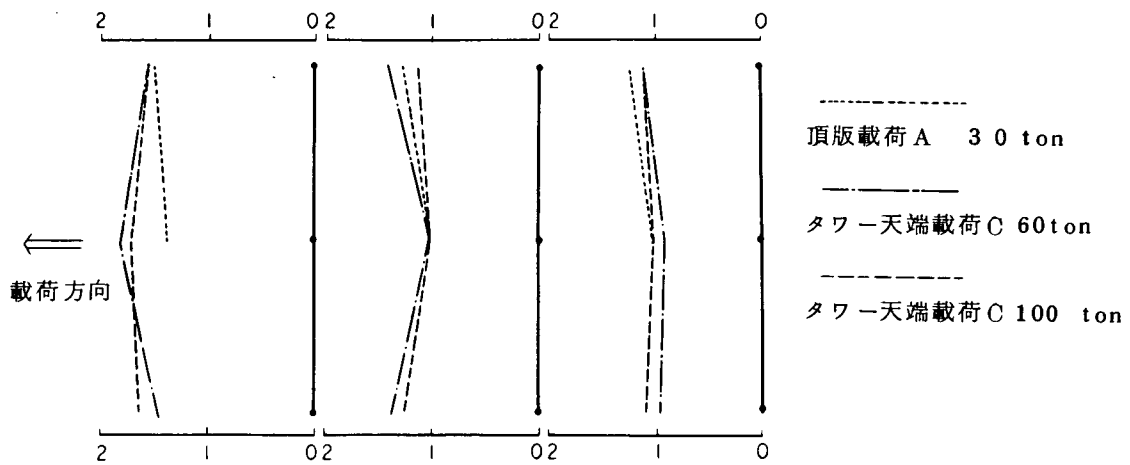


図 - 21 中央杭を基準 (= 1) とした各杭の横方向地盤反力係数比

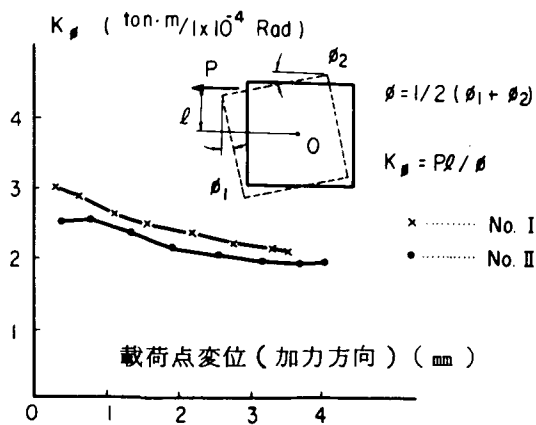


図 - 22 頂版偏心水平載荷
変位 - 頂版天端ねじりばね定数 K_ϕ

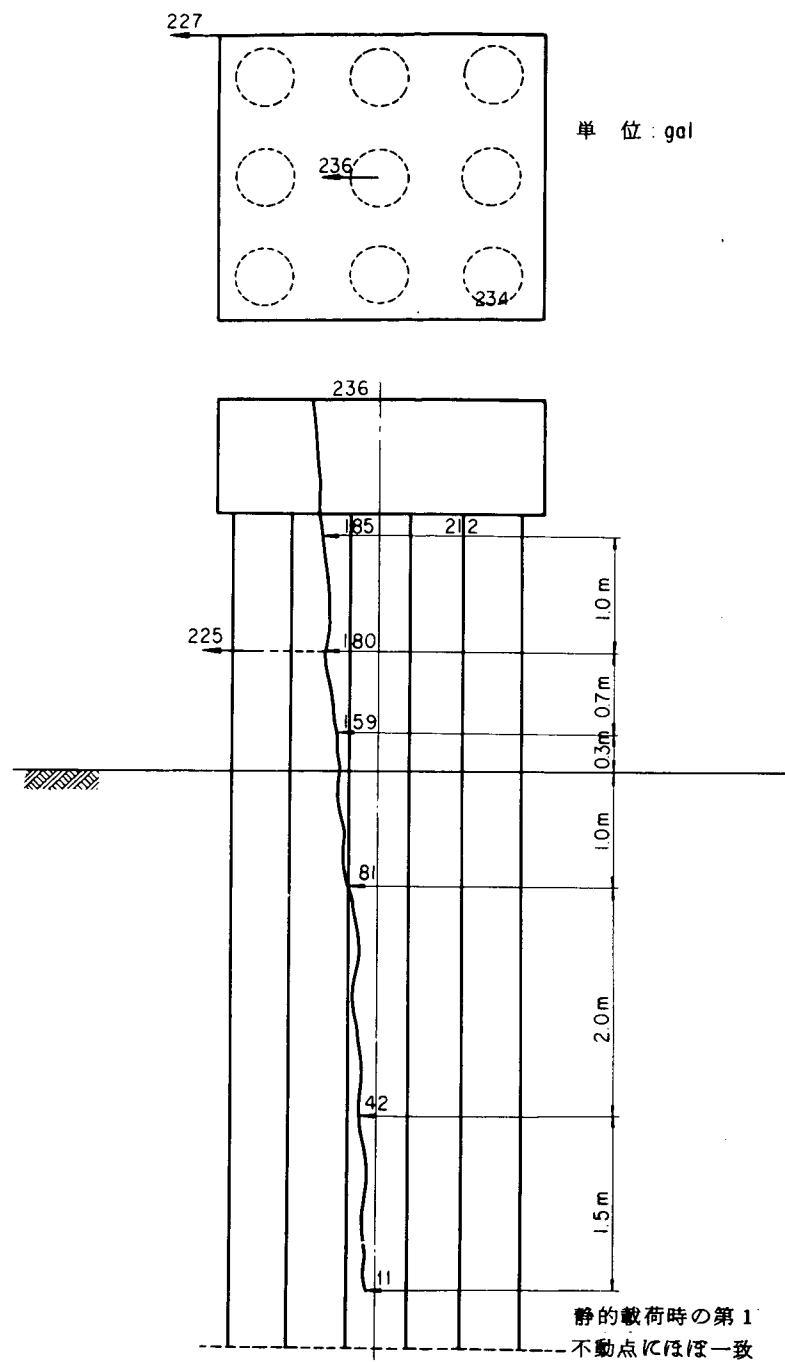


図-23 タワー天端位置自由振動加度モード

DESIGN OF PILE FOUNDATION SUBJECTED TO LATERAL LOAD



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Keiichi Komada and
Asao Yamakawa
Public Works Research Institute
Ministry of Construction

1. Introduction

Since Japan is subject to frequent earthquakes, it is necessary to have due consideration to earthquake effect in designing structures. This is specifically stipulated under Paragraph 1-8-1, "Kinds of Load" in the Volume I "Common Rules" of Japan's Highway Bridge Specification (Technical Standards of Ministry of Construction), and further, Volume V of said specification sets forth, under the title "Specification for Earthquake Resistant Design", the practical details including the design seismic coefficient, etc.

Design of sub-structures of highway-bridge is performed in compliance with the Specification's Volume IV "Specifications for Sub-structure Design". As the provisions relating to "Design of Pile Foundation" in this Bridge Specification were established in 1964 and have become rather obsolete, their amendment and revision are now being carried out in response to the progress of technology and accumulation of field and laboratory data.

This report is aimed to take up and argue the basic idea and method for design by selecting mainly from among several items studied for the revision such matters relating to design of pile foundations subjected to lateral load such as seismic load. Although these items are yet in the process of study, the important part of them is considered to be

incorporated into the provisions of said "Highway Bridge Specification". Consequently, the text of this report is to indicate a fundamental principle of Japan's highway bridge design of today.

The construction of this report and the general outline and the significance of each chapter are given as follows:

For the "design of piles," there are introduced the conventional method and the rigorous method of which the former is to solve the calculation of the pile's head reaction and displacement by merely the balance of three forces, while the latter is to consider the displacement. For usual foundations, the simple conventional method may be sufficient, but as this may cause in some cases some large errors, its application limit is indicated under a suggestion of applying the rigorous method in such cases. Generally, piles are considered sufficiently long. But considering a case where the piles are short and it is necessary to apply the former method by partially revising it, a description is given how to handle short piles.

For the "calculation of lateral coefficient of ground reaction," there is introduced a method to deduce from the results of soil tests the lateral coefficient of ground reaction which is to be applied to the analysis of the stress and displacement of the pile with the aid of the theory of beam on an elastic body.

This method was propounded on the basis of the experiments performed to obtain the correlation between the soil properties and the results of about 150 lateral load tests.

For the "Design of Pile Head", a design method for connecting pile's head with footing is described. There has been no actually practicable design method for rigidly connecting the head with the footing. But the Public Works Research Institute of the Ministry of Construction, desiring

to secure the rigidity and strength of the connections, issued a design standard, and in order to test and verify the reasonability of the standard, it carried out a series of load test of nearly a full-size scale. In this chapter, there is given a description of our basic concept of the design process of the connection of a pile with the footing and the results of said experiment.

The chapter entitled "Behavior of Pile Foundation with Free Length" describes the lateral resistance of a pile foundation having a free length and the result of the vibration test.

For the usual pile foundation of which footings are buried in the ground, there was made a plenty of vibration tests in the past, and also through the experiences of actual earthquakes, its earthquake resistance has been already verified. However, there has been hardly any appreciable experiment undertaken in respect of any pile foundation with free length since it is entirely a new type of foundation. This chapter, therefore, is dedicated to the presentation of the results of an experiment of nearly a full-size scale performed by the Public Works Research Institute.

2. Design of Pile

2-1. Calculation Method of Pile Head Reaction and Displacement

There are two methods of calculation such as the "Rigorous Method" and the "Conventional Method" for the reaction and displacement which may occur at the head section of each pile against the external forces (V_o , H_o , M_o) working upon the footing in the foundation where the pile heads, as shown in Fig. 1, are rigidly fixed. Provided, however, in either case, the footing is handled as a rigid body and the ground as being elastic both in the axial direction and the lateral direction of the pile. Here, the presented calculation method is applicable only in the case where

the piles are sufficiently long and are deemed to have an infinite length.

The applicable extent of this calculation method and the calculation method for short piles are presented in Paragraph 2-2.

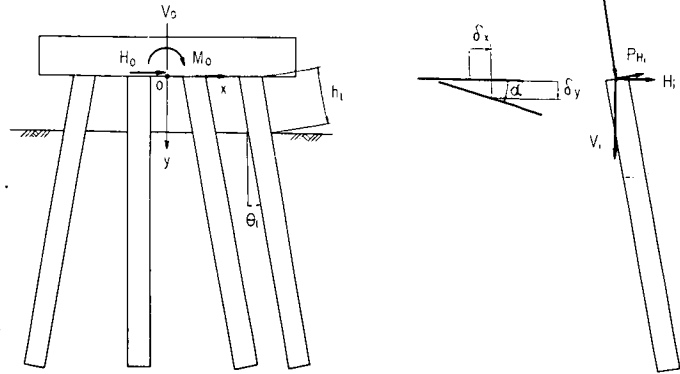


Fig. 1 Calculation model of pile foundation

(1) Rigorous Method

Taking the lateral displacement of the footing as δ_x , vertical displacement as δ_y and the rotary angle as α , these will be given by the solution of the following three-dimensional simultaneous equations:

$$\begin{cases} A_{xx} \delta_x + A_{xy} \delta_y + A_{x\alpha} \alpha = H_0 \\ A_{yx} \delta_x + A_{yy} \delta_y + A_{y\alpha} \alpha = V_0 \\ A_{\alpha x} \delta_x + A_{\alpha y} \delta_y + A_{\alpha\alpha} \alpha = M_0 \end{cases} \quad (1)$$

Here:

$$A_{xx} = \sum_i (K_1 \cos^2 \theta_i + K_v \sin^2 \theta_i)$$

$$A_{xy} = A_{yx} = \sum_i (K_v - K_1) \sin \theta_i \cos \theta_i$$

$$A_{x\alpha} = A_{\alpha x} = \sum_i \{ (K_v - K_1) x_i \sin \theta_i \cos \theta_i - K_2 \cos \theta_i \}$$

$$A_{yy} = \sum_i (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i)$$

$$A_{y\alpha} = A_{\alpha y} = \sum_i \{ (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i) x_i + K_2 \sin \theta_i \}$$

$$A_{\alpha\alpha} = \sum_i \{ (K_v \cos^2 \theta_i + K_1 \sin^2 \theta_i) x_i^2 + (K_2 + K_3) x_i \sin \theta_i + K_4 \}$$

$$K_1 = \frac{12EI\beta^3}{(1+\beta h)^3 + 2}$$

$$K_2 = K_3 = K_1 \frac{1 + \beta h}{2\beta}$$

$$K_4 = \frac{4EI\beta(1+\beta h)^3 + 0.5}{1 + \beta h(1+\beta h)^3 + 2}$$

$$\beta = \sqrt[4]{\frac{K_H D}{4EI}}$$

D: Pile diameter (m)

EI: Pile's bending rigidity (kg.cm²)

K_H: Lateral coefficient of ground reaction for a pile (kg/cm³)

K_V: Axial coefficient of ground reaction of a pile (kg/cm)

When δ_x , δ_y and α have been sought from the equation (1), the pile's axial force P_{Ni} working upon each pile head, lateral force P_{Hi} , and the pile head moment M_i can be calculated by the following equations:

$$\begin{cases} P_{Ni} = K_V \{ \delta_x \sin \theta_i + (\delta_y + \alpha x_i) \cos \theta_i \\ P_{Hi} = K_1 \{ \delta_x \cos \theta_i - (\delta_y + \alpha x_i) \sin \theta_i \} - K_2 \alpha \\ M_i = -K_3 \{ \delta_x \cos \theta_i - (\delta_y + \alpha x_i) \sin \theta_i \} + K_4 \alpha \end{cases} \quad (2)$$

(2) Conventional Method.

Considering the designing convenience, it is possible to perform an approximation calculation by the following expressions under a certain extent of application:

f) In the case of vartical piles alone:

$$\begin{cases} P_{Ni} = \frac{V_0}{n} + \frac{M_0 + \lambda H_0}{\sum_i x_i^2} x_i \\ P_{Hi} = \frac{H_0}{n} \\ M_i = -\lambda P_{Hi} \end{cases} \quad (3)$$

$$\left\{ \begin{array}{l} \delta_x = \frac{H_0}{n K_1} \\ \delta_y = \frac{V_0}{n K_v} \\ \alpha = \frac{M_0 + \lambda H_0}{K_v \sum_i x_i^2} \end{array} \right.$$

Here:

$$\lambda = \frac{1}{2} \left(h + \frac{1}{\beta} \right)$$

ii) In the case of involving skew piles:

In reference of Fig. 1, it is as follows:

$$\left\{ \begin{array}{l} V_i = \frac{V_0}{n} + \frac{M_0 + \lambda H_0}{\sum x_i^2} x_i \\ H_i = V_i \tan \theta_i + \frac{\sec \theta_i}{\sum_i \sec \theta_i} (H_0 - \sum_i V_i \tan \theta_i) \\ P_{Ki} = V_i \cos \theta_i + H_i \sin \theta_i \\ P_{Hi} = -V_i \sin \theta_i + H_i \cos \theta_i \\ M_i = -\lambda P_{Hi} \\ \delta_x = \frac{H_0}{\sum_i (K_1 \cos^2 \theta_i + K_v \sin^2 \theta_i)} \\ \delta_y = \frac{V_0}{n K_v} \\ \alpha = \frac{M_0 + \lambda H_0}{K_v \sum_i x_i^2} \end{array} \right. \quad (4)$$

(3) Application Limit of Conventional Method

In case that the pile foundation is formed with only vertical piles in symmetrical arrangement, both the rigorous method and the conventional

method perfectly concur with each other in the pile-head reaction and lateral displacement respectively, in the case of $K_v = \infty$. Meanwhile, in the case of $K_v = 0$, the behavior of each pile becomes the same as in a single pile. In the actual pile foundation, it maintains an intermediary state between the two, and by the non-dimensional parameter ϕ defined by the following expression, the rigidity ratio between the vertical direction and the horizontal direction is expressed:

$$\phi = \frac{K_v \sum_i x_i^2}{K_v \sum_i x_i^2 + \frac{n E I \beta}{1 + \beta h}} \quad [0 \leq \phi \leq 1] \quad (5)$$

Here; N: Number of piles.

By employing this parameter ϕ , the solution (defining it as f) by the rigorous method of both the pile-head reaction and displacement can be simply expressed by the following expression:

$$f = f_{(K_v=0)} + \phi \{ f_{(K_v=\infty)} - f_{(K_v=0)} \}$$

Here; $f_{(K_v=0)}$ is the solution when it is $K_v=0$, and is the calculated value of a single pile. $f_{(K_v=\infty)}$ is the solution when it is $K_v=\infty$, and is the calculated value by the conventional method. Namely, the parameter ϕ becomes closer to 1 as the vertical direction rigidity of a pile foundation as a whole grows larger comparing the lateral direction rigidity. Consequently, the gap between the rigorous method and the conventional method narrows down. From the results of preliminary calculation of various types of foundations, it has been ascertained that if it is $\phi > 0.95$, the gap between the two calculation methods is more or less below 20%.

In the case where skew piling is involved, it is difficult to make judgment of the applicable extent of such a simple parameter as mentioned above, but the similar thing may be said as to its trend that when the

vertical direction rigidity is small and the oblique angle is substantially great, the errors arising from the conventional method become larger.

2-2. Design of Short Piles

As practiced generally in the design of piles, viewing a pile as a beam on an elastic body, the pile's displacement y in the model as presented by Fig. 2 can be expressed by the following 4' order differential equations.

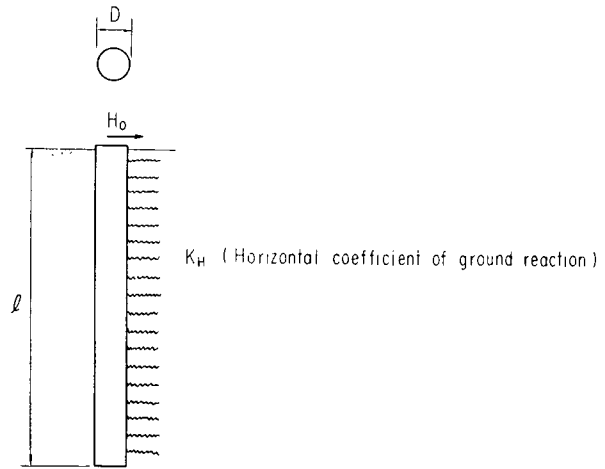


Fig. 2 Calculation model of single pile

$$EI \frac{d^4 y}{dx^4} + K_H D y = 0 \quad (6)$$

Solving this with inclusion of the boundary condition, the pile head displacement rate δ at the time when the lateral force H_0 affected the pile head is expressed by,

$$\delta = \frac{1}{\alpha} \frac{2 \beta H_0}{K_H D} \quad (7)$$

Here: $\frac{2 \beta H_0}{K_H D}$ is the pile head displacement rate in the case the pile length l is infinite, and the coefficient α is the correction coefficient for the pile because of its being of a limited length, while βl alone is expressed by a function. On the other part, as the pile head displacement rate when the pile is regarded as a rigid body becomes $\delta = \frac{4 H_0}{K_H D l}$, the coefficient in the equation (7) is $\alpha = \frac{\beta l}{2}$.

Fig. 3 represents the relation between α and βl , and it can be learned from this that if it is $\beta l > 3$, it is $\alpha \doteq 1$ and is close to the

solution of an infinite length, and that if it is $\beta \ell < 1$, it is $\alpha \doteq \frac{\beta \ell}{2}$ and is close to the solution of a rigid body.

The above description is focused only on the pile displacement under the influence of a lateral force. But even in other cases, the similar

tendency is also noted. After all, as to the lateral behavior of a foundation in the ground, it is possible to make the following classification on the basis of a design principle:

$\beta \ell \geq 3$ Elastic body. infinite length

$1 < \beta \ell < 3$ Elastic body. limited length

$\beta \ell \leq 1$ Rigid body.

A pile in-between the "rigid body" region and the "elastic limited length" region is called generally a "short pile," and the boundary conditions at the end-point of the pile must be taken into consideration for the calculation.

Fig. 4 indicates the outline of a lateral load test performed with use of $\phi 2m$, ℓ 17m PC Well. In this case, with $\beta \ell = 2$, the pile belongs to the region of "short piles," and its end-point is driven about 5m deep into the gravel ground.

Fig. 5 represents the comparative values derived by calculating the measured oblique angle distribution and the boundary conditions at the pile's end-point by changing them in three types of free, hinged and fixed. As shown in this case, when the end-point is substantially driven into the gravel layer ground, the calculation values and the measured values

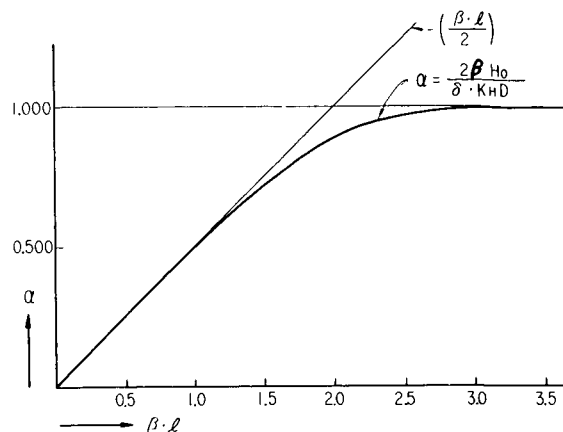


Fig. 3 Relationship between α and $\beta \ell$

fairly well correspond with each other when the end-point is assumed as a hinge.

Fig. 6 illustrates the influence of the support conditions at the end-point of the pile upon the flex distribution of the pile and the bending moment distribution.

3. Lateral Coefficient of Ground Reaction (K_H).

The greatest factor to control the pile's lateral resistance is the lateral resistance of the ground itself. The load-deformation characteristics of the ground are generally non-linear, and moreover, when a load is applied by or through such a flexible body as a pile, it is extremely difficult to determine a definite yield-point since a breakdown takes place one after another from the upper section as load is applied. Because of this, in designing, the ground is assumed to be elastic taking into consideration the precise data of the ground survey, and the pile is deemed as a beam supported on this elastic body, and the calculation is made on this basis.

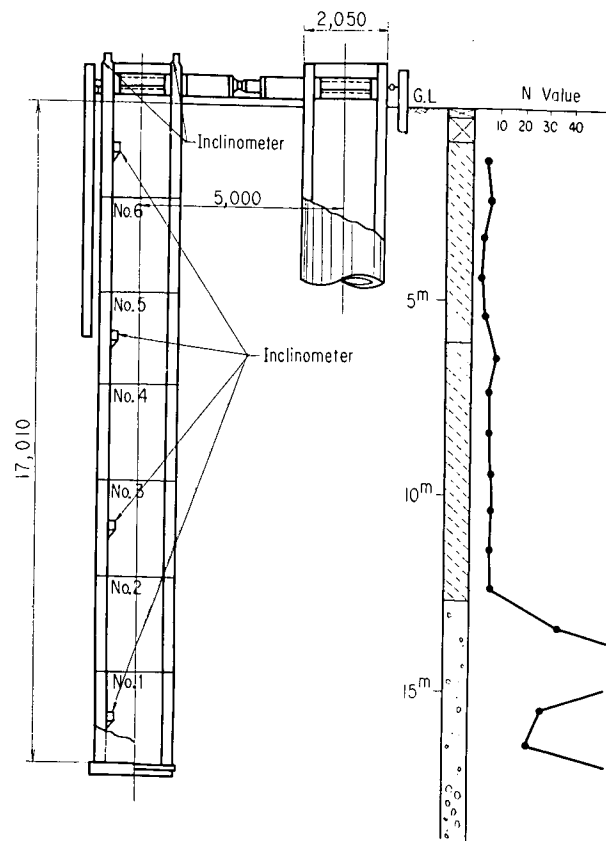


Fig. 4 Horizontal loading test of PC pile with large diameter

Thus, the design load is determined from the allowable displacement rate and the pile's permissible stress. In this case, as the ground naturally is non-linear, the standard ground surface displacement rate is determined and a virtual free spring constant against so determined displacement rate is employed. From this comes the lateral coefficient of ground reaction

K_H . In order to find out this deduction, a number of experiments have been undertaken.

Firstly, in order to investigate the effect of the size of the loading plate upon the lateral coefficient of ground reaction, test pits were made in sand-ground and loam-ground, and using circular loading plates of 30cm, 60cm, 90cm and 120cm diameters respectively, a series of horizontal plate loading tests were

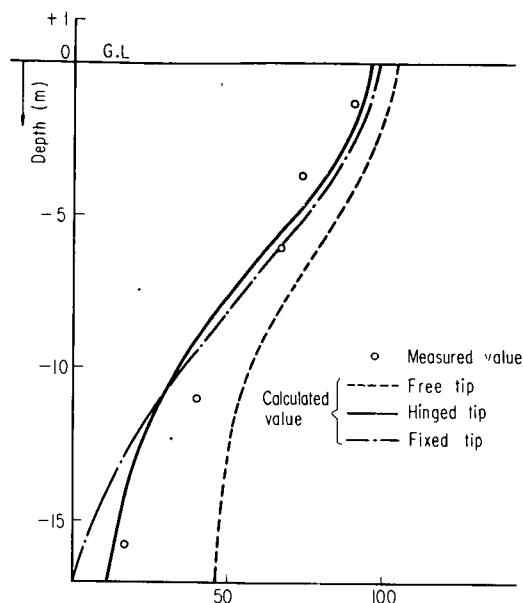


Fig. 5 Distribution of angle of inclination (Comparison with calculated value)

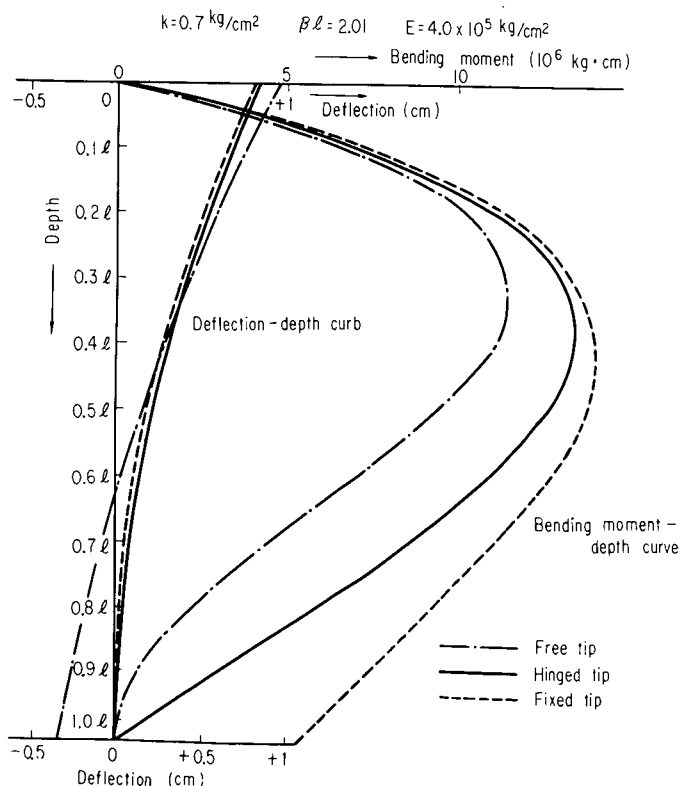


Fig. 6 Difference due to supporting condition at pile tip

carried out.

Fig. 7 shows that the value of various diameter is expressed in a ratio with K_{30} when K_H in the case of $D=30\text{cm}$ is placed as K_{30} . It was confirmed that K_H declined in an inverse proportion to the loading plate raised to $3/4$ power regardless of the ground depth or pile buried depth.

That is:

$$\frac{K_H}{K_{30}} = \left(\frac{D}{30} \right)^{-\frac{3}{4}} \quad (8)$$

The virtual deformation coefficient obtained from the plate loading test by use of 30cm-plate loading plates was assumed as K_{30} which was considered as the performance value, and on the other part, in order to examine the inter-relation in various ground investigation methods, a comparative experiment was conducted for E_{30} against the deformation coefficient E_p obtained by a boring hole circular loading test, the deformation coefficient E_c obtained by either mono-axial or tri-axial test by use of sample materials,

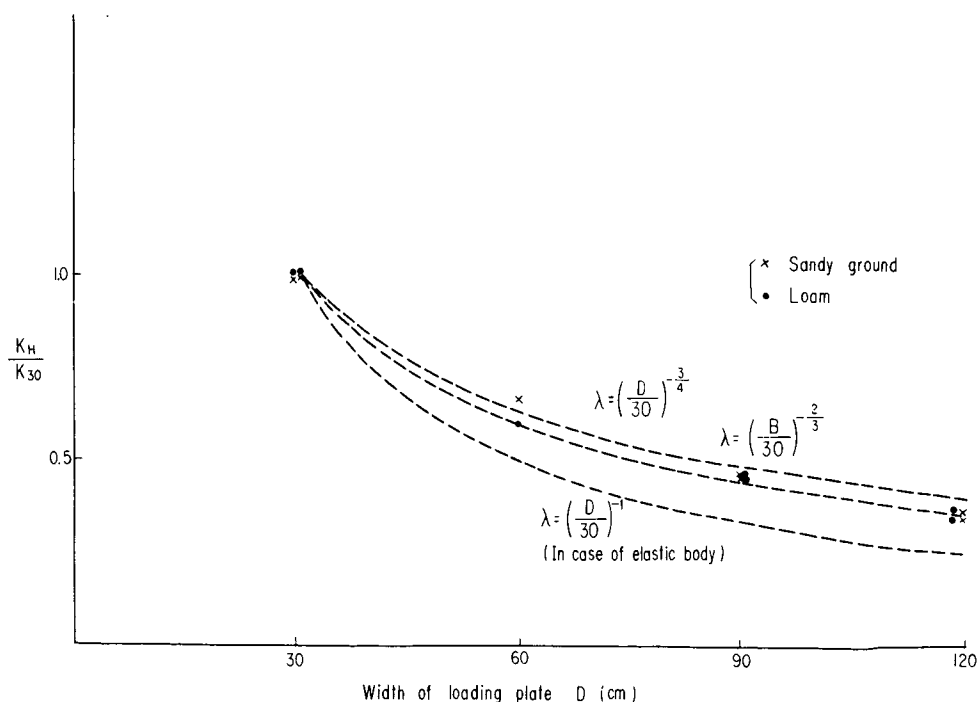


Fig. 7 Relationship between loading plate and coefficient of ground reaction

and the standard penetration test value N , and from the results as shown by Fig. 8 and Fig. 9, an approximation experimental expression as shown below was obtained:

$$E_{30} = 4 E_P = 4 E_C = 28 N \quad (9)$$

By adding a proportional constant which is experimentally determined from a pile loading test to the expression of relation between (8) and (9), the following expression can be proposed as an estimation expression of the lateral coefficient of ground reaction for a pile K_H :

$$K_H = \frac{E_{30}}{4.8 D^{\frac{3}{4}}} = \frac{E_P}{1.2 D^{\frac{3}{4}}} = \frac{E_C}{1.2 D^{\frac{3}{4}}} = \frac{7 N}{1.2 D^{\frac{3}{4}}} \quad (10)$$

Fig. 10 represents the comparison between the results of a single pile loading test performed on a loam ground and the expression (8), and Fig. 11 illustrates the relation between $7N/K_H$ and the pile's diameter (D) based on the expression (10) by collecting the data on 150 cases of actual pile horizontal loading tests throughout Japan. Here, for K_H , the value in the case of 10mm of ground surface displacement was employed. For the standard penetration test value N , the mean value up to the depth from the ground surface $1/\beta$ ($\beta = \sqrt[4]{\frac{K_H D}{4 E I}}$) was employed.

4. Design of Pile Head

Since it is necessary that the load applied to the super-structure or the sub-structure of a pile should be transferred without failure to the foundation structure, the connection of the pile body to the footing structure is extremely important. Especially in the case of a pile foundation, as the cross section will be the portion which will make a sudden change, a smooth transfer of load is imperative.

At present, for the sub-structure of a highway bridge, the connection

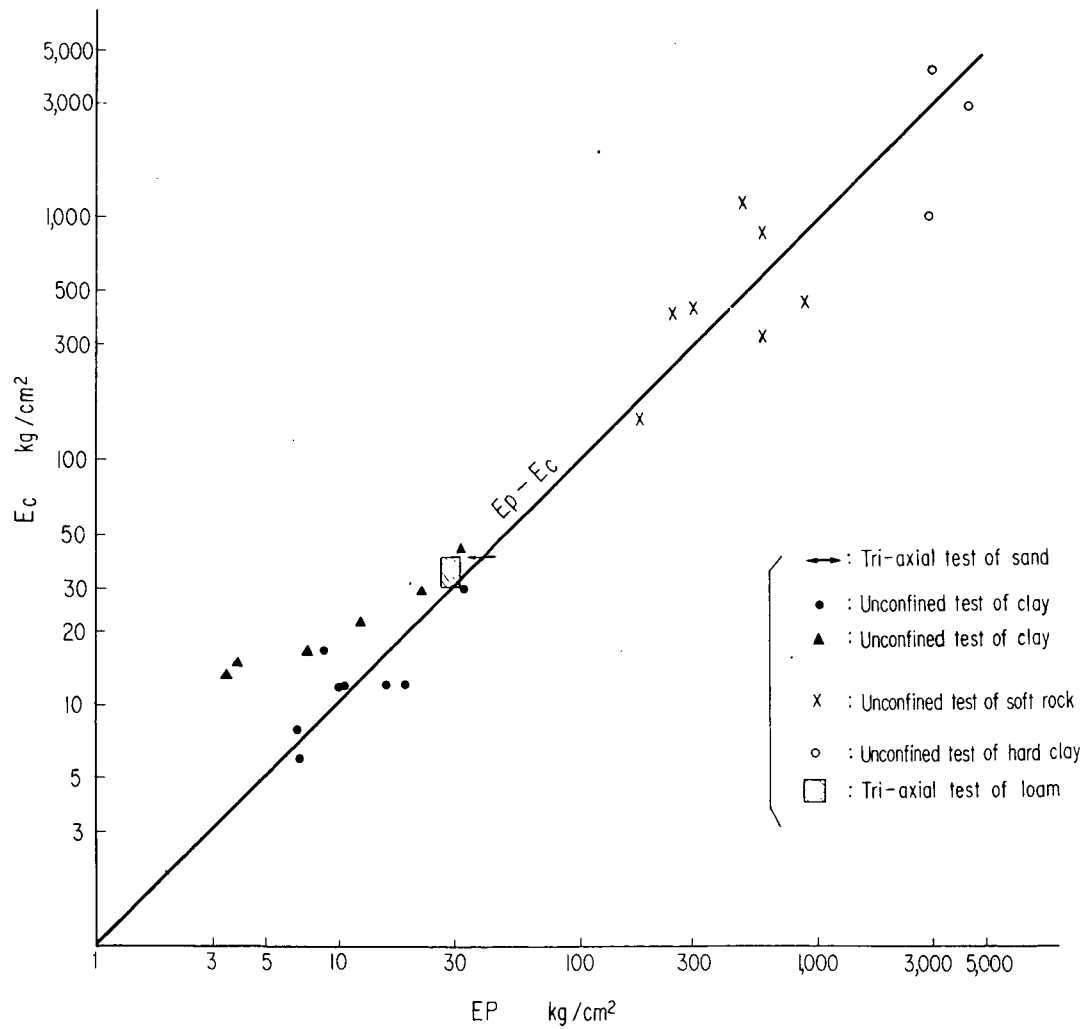
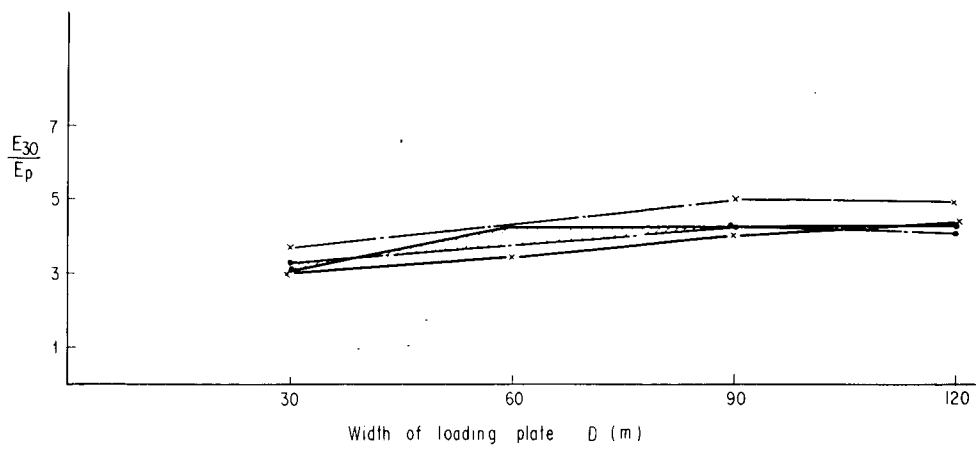


Fig. 9 Relationship between E_p and E_c

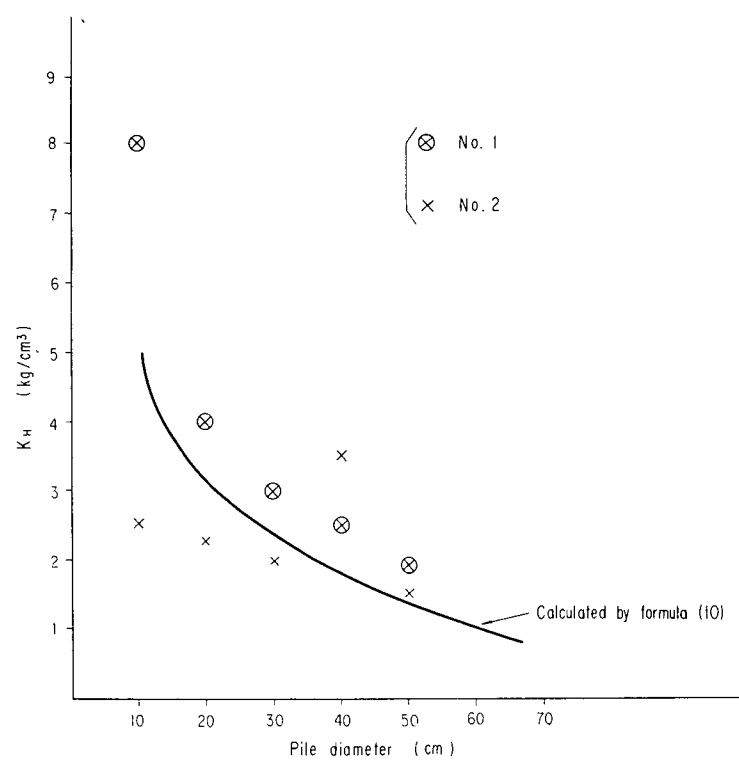


Fig. 10 Relationship between pile diameter and K_H

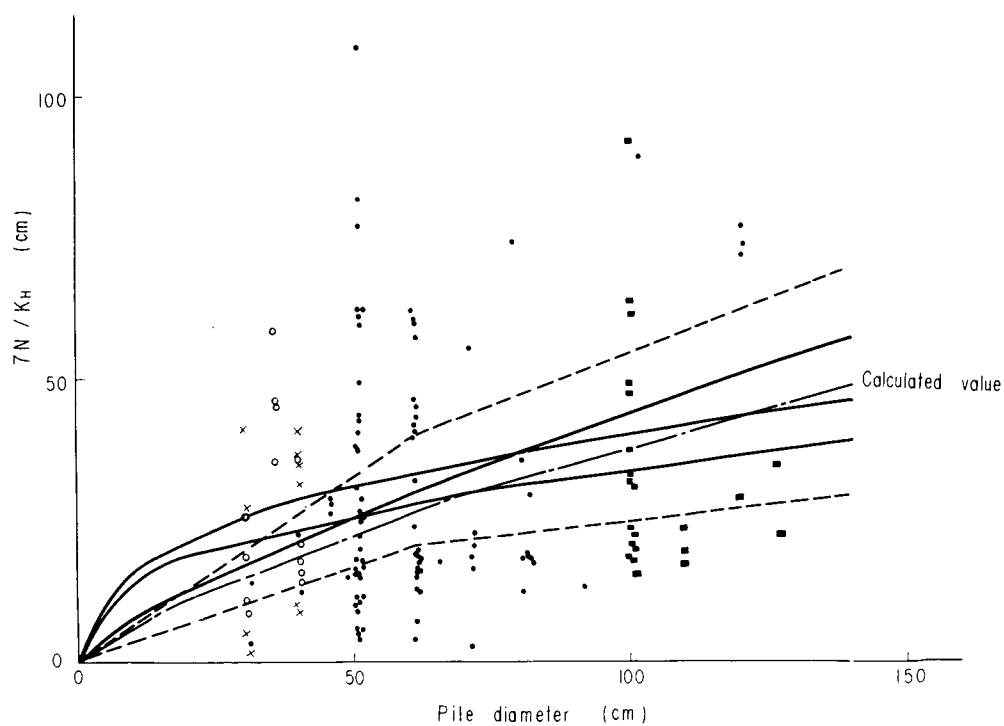


Fig. 11 Pile diameter and K_H

of piles and footing is regulated to be designed in principle as a rigid structure by use of either of the following two methods:

Method A: By piles being buried into the footing to a certain designated depth, the combined structure is to resist the pile head confining moment by the buried portion of the pile. This method is applicable to steel pipe piles, PC and RC piles. The details of connection shown by Fig. 12, (1) and (2) belong to this method.

Method B: This is to resist the pile head confining moment by mainly reinforcing the piles, although the length of the buried pile into the footing is short. This method is applicable to any of steel pipe piles, PC piles, RC piles or cast in-place reinforced concrete piles. The methods shown in Fig. 12 (3) - (5) belong to this method.

The design method is specified by each kind of pile and by the methods B and A. For example, a design method for a steel pipe pile which is connected by use of Method B is introduced as follows:

- (1) The connection of the pile and the footing is designed as a rigid structure, and the pile's head should be designed so as to resist to all of the punching, pull-out force, lateral force and the moment.
- (2) The length of pile head to be buried into the footing shall be 10cm.
- (3) The vertical bearing stress of the footing concrete shall be (refer to Fig. 13):

$$\sigma_{cv} = \frac{P}{\frac{\pi}{4} D^2} \leq \sigma_{ca}$$

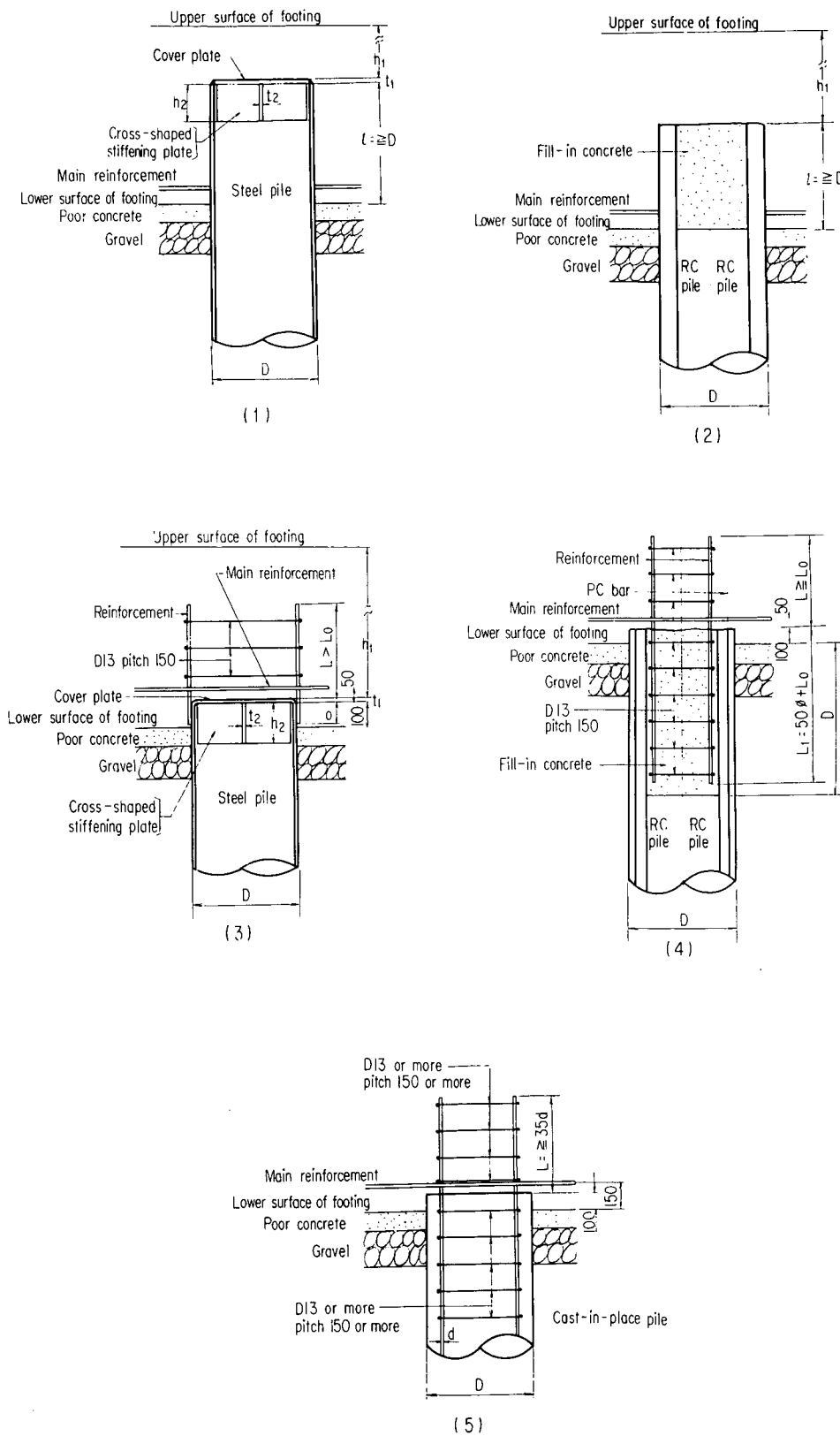


Fig. 12 Connection between pile and footing

- (4) The lateral bearing stress of the footing concrete shall be:

$$\sigma_{ch} = \frac{H}{D \ell} \leq \sigma_{ca}$$

- (5) The pull-out shear stress of the footing concrete shall be:

$$\tau = \frac{P}{\pi (D + h_1) h_1} \leq \tau_a$$

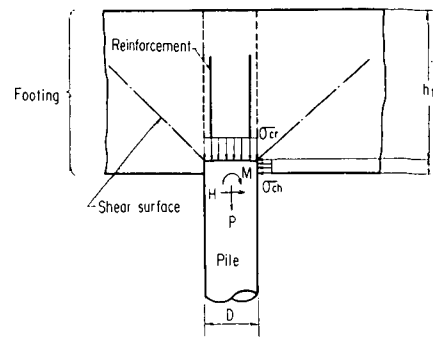


Fig. 13 Design model to method B

- (6) The cover plate and the cross reinforced plate:

$$t_1 = t_2 = 22\text{mm} \quad h_2 = 30\text{mm}$$

Provided that these are applicable to piles of not more than 1m diameter.

t_1 : Thickness of the cover plate. (cm)

t_2 : Thickness of the cross reinforced plate. (cm)

h_2 : Height of the cross reinforced plate. (cm)

- (7) Hypothetical reinforced concrete cross section stress.

In the case where either the punching force P and the moment M or the pull-out force P_t and the moment M affect, a cross section of a reinforced concrete is assumed and the stress of concrete and the reinforcement is studied.

- (8) Anchoring of reinforcement.

- i) Shearing stress at the welded section.

$$\tau_s = \frac{\sigma_{sa} A_{st}}{1.4 \lambda \ell_0} \leq \tau_{sa}$$

σ_{sa} : Allowable tensile stress of the reinforcement (kg/cm^2)

A_{st} : Cross section of the reinforcement (cm^2)

λ : Leg length of the fillet weld (cm)

ℓ_0 : Weld length (cm)

- ii) Anchor length of the reinforcement

$$L_0 = \frac{\tau_{sa} A_{st}}{\tau_{0a} \cdot u}$$

τ_{0a} : Allowable bond stress of concrete (kg/cm^2)

u : Hoop length of the reinforcement (cm)

- (9) As to the piles at the end of the footing, the lateral pull-out shearing stress will be studied.

Then, the results of the model test on steel pipe piles which are connected by Method B are introduced as follows:

Fig. 14 represents the model's form and measurement.

As shown by Fig. 15, the pile head connection has a 10cm built-in and is reinforced with 8 steel bars of 16mm diameter. On the pile's head, there is provided a steel plate cover of 16mm thick and 16mm-ring reinforcement is also provided around the steel pipe pile to resist the circumferential tensile stress.

The vertical load is 90 tons maximum, and the stress distribution on the footing concrete and the pile head cover plate under this maximum load is shown by Fig. 16, and the comparison of the actually measured stress value and the calculated value is given in Fig. 17. Provided, however, that the calculated value of the cover-plate stress is sought as follows assuming that the entire bearing stress would

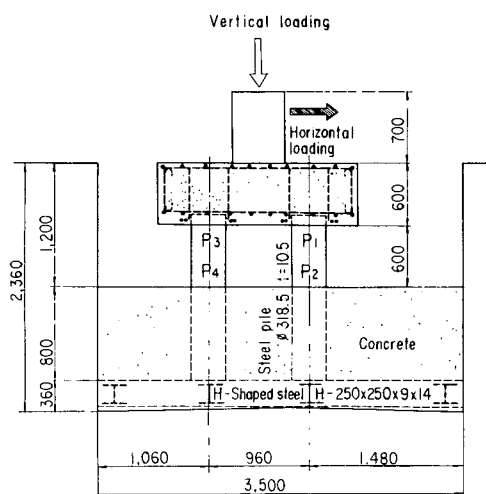


Fig 14 Shape and size of model

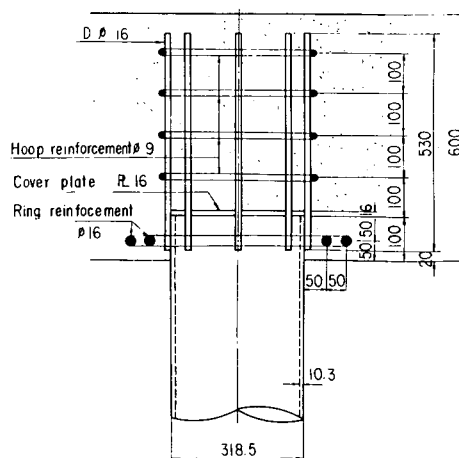


Fig.15 Detail of connection

affect upon the cover-plate:

$$\sigma = \frac{99}{320} \sigma_c \left(\frac{D}{t} \right)^2$$

σ_c : Bearing stress (kg/cm^2)

D : Pile diameter (cm)

t : Pile thickness (cm)

However, the transfer of vertical load upon the pile actually concentrates at the wall thickness, and because of this, the actually measured value of the stress at the cover-plate is smaller than the calculated value.

Then, removing the vertical load, a lateral load of 70 tons maximum was applied. Fig. 18 shows the comparison of the actually measured stress values and the calculation values with respect to P_1 and P_4 piles.

From this result, it is indicated that there exists a definite gap in the behaviors of P_1 on the punching side and P_4 on the pull-out side. It is known from the bearing stress distribution that the entire area of P_1 is compressed, while with P_2 , there is some rotation occurred at the built-in section. This clearly means that the confinement on the part of P_4 is rather weak, and that to

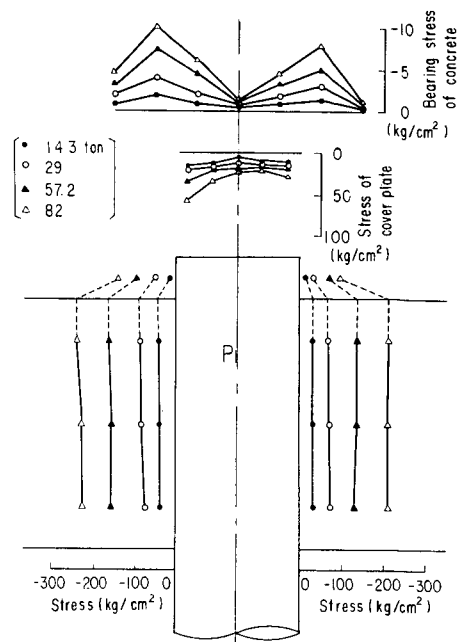
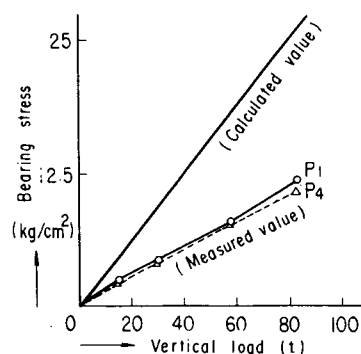
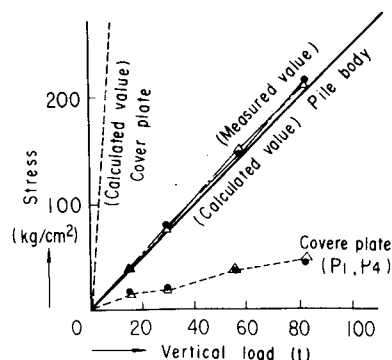


Fig 16 Distribution of stress

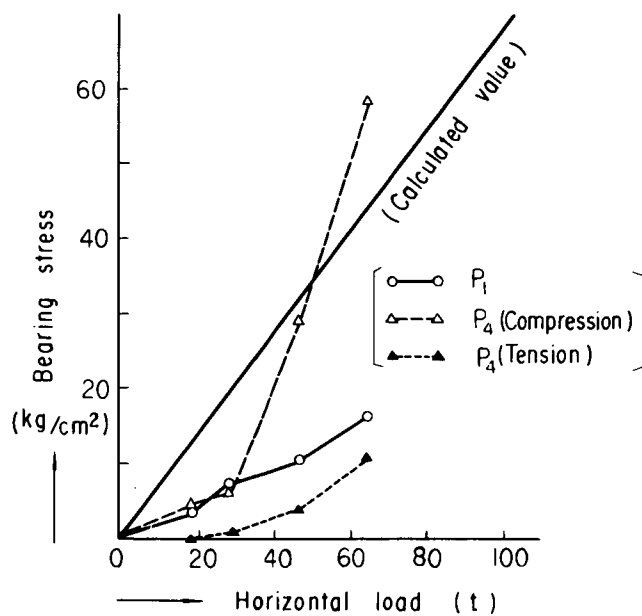


(a) Bearing stress of footing concrete

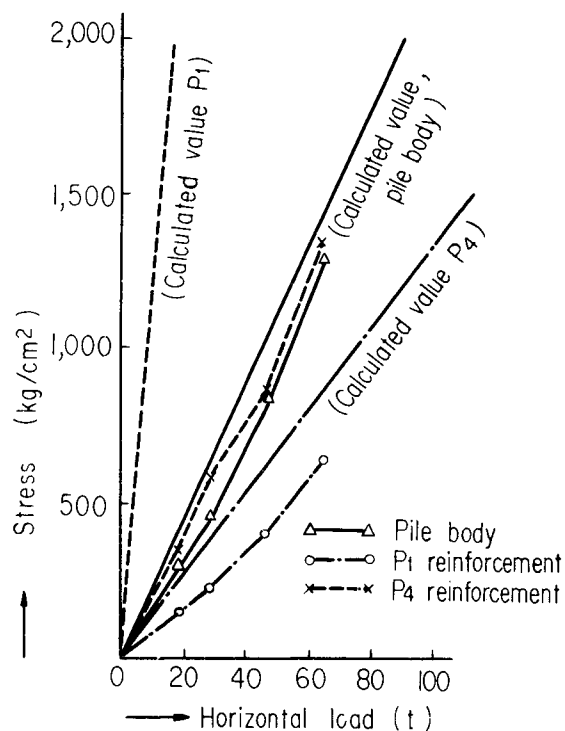


(b) Stress of pile body and cover plate

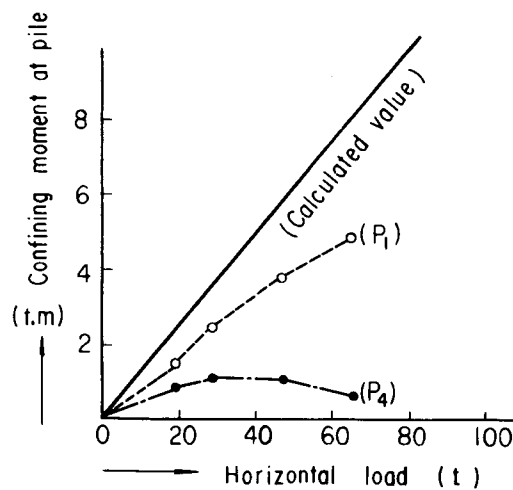
Fig.17 Comparison between measured value and calculated value



(a) Bearing stress of footing concrete



(b) Stress of stiffening reinforcement



(c) Confining moment at pile

Fig. 18 Comparison between calculated value and measured value

make the pile head to be perfectly a fixed state is extremely difficult.

However, as a whole behavior pattern, the presently established pile head connecting method has a sufficient design strength, and the stress occurring inside the footing is within more safe values than the general calculation values.

5. Behavior of Pile Foundation with Free Length

The multiple-pillar foundation of rigid structure which is formed by groups of large diameter columns or pillars which penetrate and are embedded into a designated bearing strata and their head section is rigidly held together by a top-cover plate excels in in-water workability and the cases of work-execution of this type have rapidly increasing lately. Even in the large scale projects involving highway bridges linking the mainland and Shikoku Island is included this type of foundation structure. For the purpose of examining the lateral resistance characteristics and behaviors of the pile groups with such free length, a series of experiments with use of large size models have been undertaken, and the results are given as follows:

As shown by Fig. 19, two models having different thickness of the top-cover plates were made, and using these models, a series of experiment including a lateral loading test, a free behavior test, and a forced vibration test (the top-cover plate) have been conducted. The symbols A, B and C indicate the location of the applied load and the loading direction.

Fig. 20 illustrates the relation between the average lateral coefficient of ground reaction (K_H) and the ground surface displacement obtained from the lateral loading test. By the group-pile effect, K_H declined more than in the case of a single pile, and it also fluctuated by loading position.

Fig. 21 shows the fluctuation of K_H in the ratio of the central pile which is set as the standard. The share ratio of the front line piles is about 50% larger than that of other piles.

Fig. 22 represents the torsion-spring constant in the case of an eccentric loading (loading position B). In this case, the pile's torsion resistance had 80% of the entire torsion resistance, while only a 20% was by the ground lateral reaction. With the multiple-pillar type foundation having a lower rigidity, it becomes necessary to check its torsion rate.

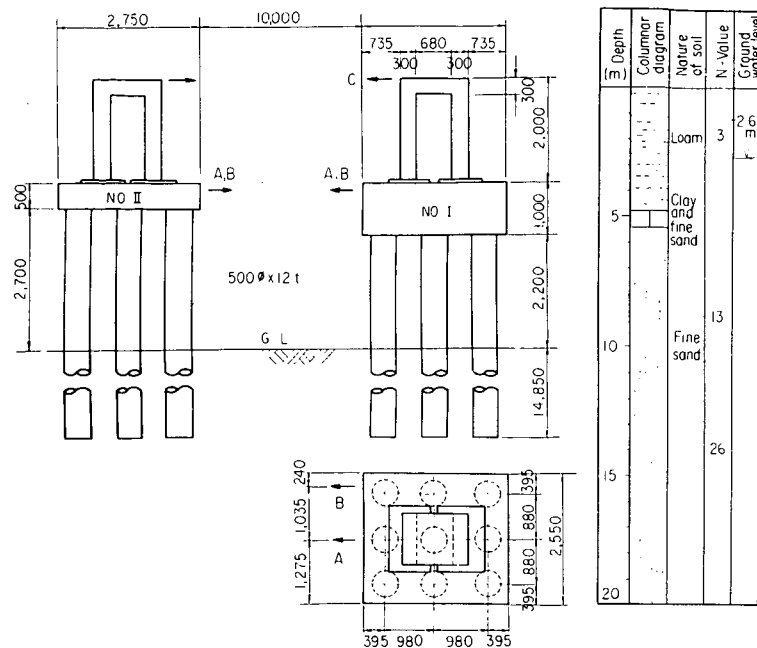


Fig. 19 Structure and dimensions of models

Ground condition

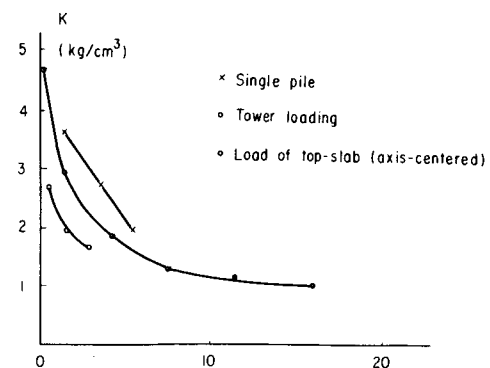


Fig. 20 Displacement of pile at the ground surface vs. lateral ground reaction coefficient K (mean)

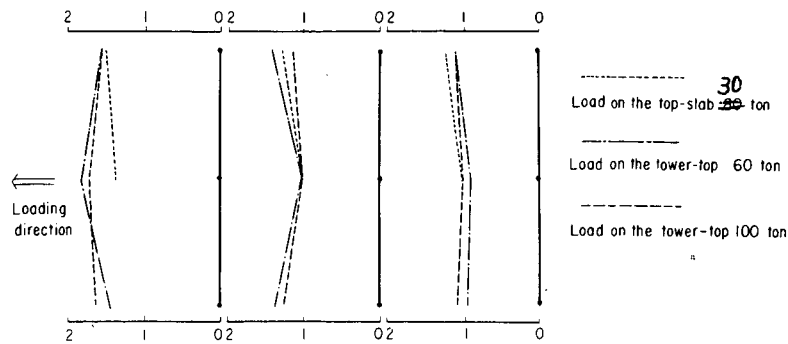


Fig. 21 Lateral ground reaction coefficient ratio of each pile on the basis of central pile (=1)

Fig. 23 shows the acceleration mode by free vibration at the tower-end position (C). Table 1 and 2 present the results of the inherent vibration frequency and the damping constant obtained from the free vibration test and the forced vibration test.

When the inherent period of the multiple-pillar foundation is sought by replacing it with a model of one degree of freedom, and taking the total mass of the ground surface as the mass which relates to vibration, calculate K_H of the ground inversely from the measured inherent period. Then, it is:

Top-cover axial free vibration (A) $\longrightarrow K=1.23 \text{ kg/cm}^3$

Tower axial free vibration (C) $\longrightarrow K=1.24 \text{ kg/cm}^3$

Forced vibration (top-cover position) $\longrightarrow K=1.08 \text{ kg/cm}^3$

Thus, these values are either comparable to or slightly lower than K_H by the static load.

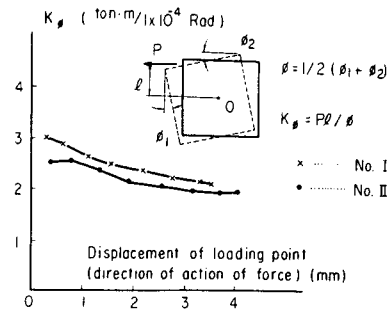


Fig. 22 Eccentric horizontal load on the top-slab, displacement vs. torsional spring constant on the top-end of the top-slab.

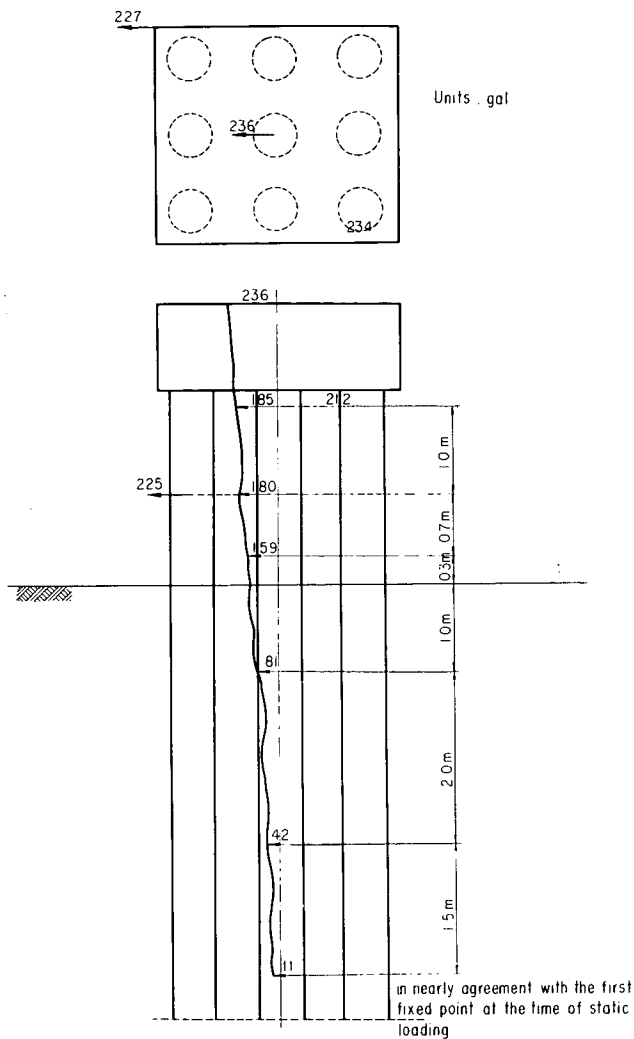


Fig.23 Acceleration made of natural vibration at the tower-top position

Table 1. Natural frequency f (c/s), damping factor h (natural vibration)

Set position of Accelerometer		Axis-centered natural vibration test at the top-slab		Eccentric natural vibration test at the top-slab		Axis-centered natural vibration test at the tower-top	
		f	h	f	h	f	h
No. I	Axial direction on the center of the top-slab	8.5	0.074	8.5	0.088	7.8	0.068
	Perpendicular-to-axis direction on the center of the top-slab	-	-	9.5	0.058	-	-
	Axial direction on the upper corner of the top-slab	8.5	0.071	8.0	0.058	7.8	0.062
	Perpendicular-to-axis direction on the upper corner of the top-slab	-	-	15.2	0.064	-	-
	Axial direction at 2m above the ground of the central pile	8.5	0.075	8.3	0.086	7.8	0.075
	Axial direction at 0.3m above the ground of the central pile	8.5	0.088	8.3	0.097	7.8	0.071
No. II	Axial direction on the center of the top-slab	9.5	0.128	10.0	0.153	8.3	0.058
	Perpendicular-to-axis direction on the center of the top-slab	-	-	18.0	0.077	-	-

Table 2 Resonance frequency

	$f' \text{ (C/S)}$
Damping factor	h
Natural frequency	$f \text{ (C/S)}$
	$f = f' \sqrt{1-2h^2}$

(No. 1, central pile, from the measured strain, at the point 1.7m above the ground)

Vibrational power	$f' \text{ (C/S)}$	h	f
0.3	7.3	0.073	7.85
0.3	7.85	0.078	7.9
0.6	7.85	0.078	7.9
0.6	7.85	0.080	7.9
1.8	7.65	0.087	7.7
1.8	7.50	0.066	7.55

Postscript

This report has taken up a number of basic items for the design of pile foundation subjected to lateral load, and dealt with the present situation of highway bridge design in Japan. As the items to be further studied for standardization relating to the design of pile foundation bearing a lateral load, there are the problems, for example, of deformation mechanism of group-pile structures. In this respect, a research work is being continued.

In respect of the design of pile-head, this report introduces only the method by reinforcement. But as an embedding or built-in method is being developed, its results may be reported in some future.