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7.02.01

IOWA DEPARTMENT OF TRANSPORTATION

OFFICE OF BRIDGE DESIGN

DESIGN CRITERIA FOR PIERS

DEPARTMENT OF TRANSPORTATION
IOWA

The following criteria for pier design are based on our interpretations of the 1977 AASHTO Bridge Specifications. It is our intention to maintain essential compliance with current AASHTO specifications, and therefore, these criteria are subject to change with changes in relevant AASHTO specifications.

Consulting Engineers who have a copy of these criteria should ascertain that their copy is current before starting work on Commission projects involving pier design.

May, 1979

C. Pestotnik
Bridge Engineer

Revision	
Date	Pages
6-1-79	entire book

DESIGN CRITERIA FOR PIERS

OFFICE OF BRIDGE - IOWA DEPARTMENT OF TRANSPORTATION

(1) Design Specification

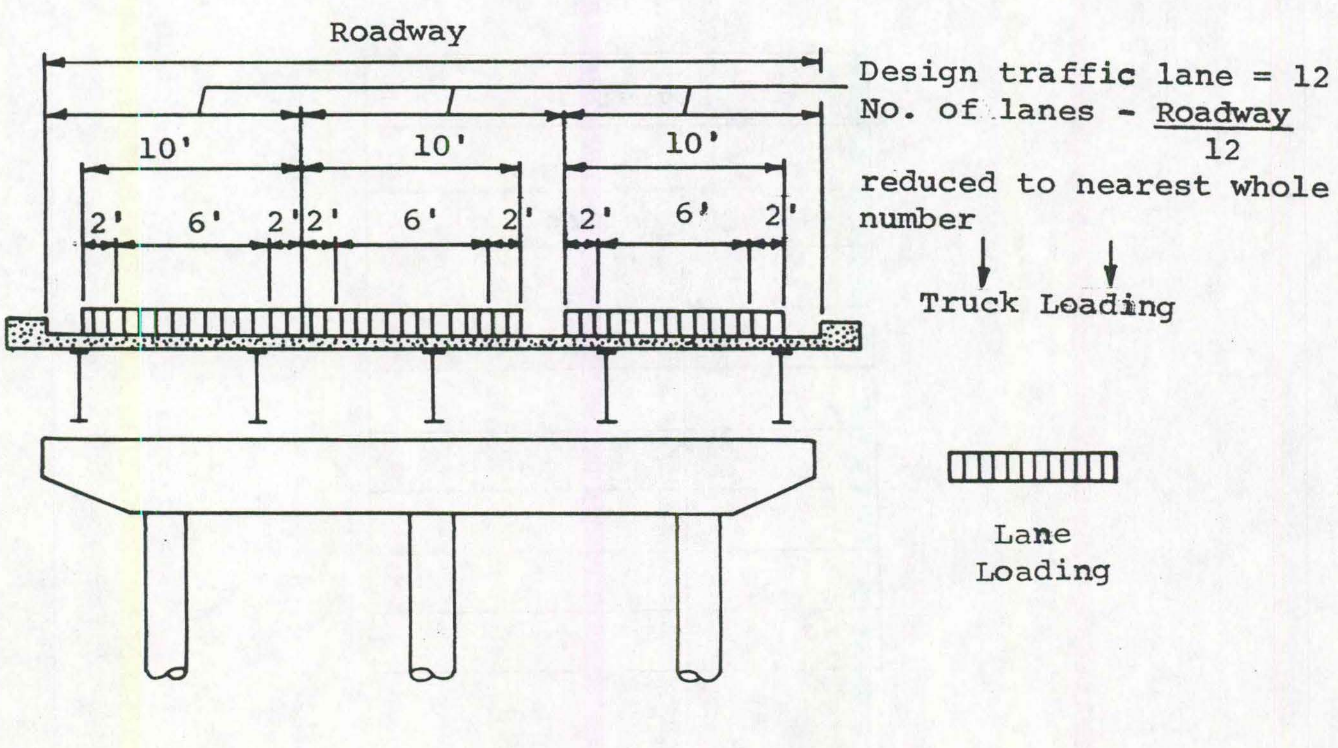
The 1977 Standard Specifications for Highway Bridges of A.A.S.H.T.O., except with the following modifications and clarifications.

(2) Live Load (L) Art. 1.2.5. & Art. 1.2.6.

(a) For live load reaction, the lane or truck loading can be placed anywhere within the design traffic lane with the exception for truck loading the minimum distance between wheel and boundary of each lane shall be 2'.

(b) For calculating beam reactions on the pier, the live load is to be distributed to each beam with the assumption that the slabs between beams are simply supported, except as modified in (2d).

(c) The live load shall be so arranged within design traffic lanes as to produce the beam reactions required to produce maximum stresses in pier.



(d) For design of cantilevers supporting one beam only the beam reactions on the cantilevered portion of the pier due to live load shall be calculated on the basis of load distribution factor as stated in Art. 1.3.1. (B). The design analysis of pier frame, shaft, footing and piling shall be in accordance with the distribution of load as stated in (2b). Cantilevers supporting more than one beam shall also be designed in accordance with (2b).

(3) Impact Load (I) Art. 1.2.12.

The impact load shall be applied to (1) the portion of pier above footing and (2) the piling in a pile bent.

The impact load shall not be applied in the design of footing and piling.

(4) Dead Load (D)

(a) For the usual bridge in regard to pier design, the entire dead load of barrier and floor slab overhang is assumed to be carried by the exterior beam. Sidewalk or other heavy loadings applied on the exterior beam or an overhanging slab are not considered "usual" and partial distribution of these loads to interior beams will be acceptable.

(b) The floor dead load is to be distributed to each beam with the assumption that the slab between beams is simply supported.

(5) <u>Longitudinal</u> <u>Force</u> <u>from</u> <u>Live</u> <u>Load</u> (LF) Art. 1.2.13

(a) LF for H20 or HS20 Loading.

$$LF\ (lbs) = \underline{N}\left[(640 \times BL) + 18{,}000\right]\ (0.05)\ (k)$$

$\underline{N}$ = number of lanes likely to become one directional in future.

BL = over-all bridge length in ft.

k = coefficient of reduction in load intensity according to Art. 1.2.9.

The c.g. of LF shall be assumed to be applied at <u>6 ft</u>. above roadway, except as modified in (8) and (9) for piers of multiple span continuous beam bridge, in (10) for piers of prestressed concrete beam bridge.

(b) Distribution of LF to piers

LF to each pier except as modified in (8) and (9) for piers of multiple span continuous beam bridge =

$$\frac{LF}{\text{Bridge Length}} \times \overline{\text{Avg.}}\ \text{Span Length}$$

The "Bridge Length" and "Avg. Span Length" are defined in (8).

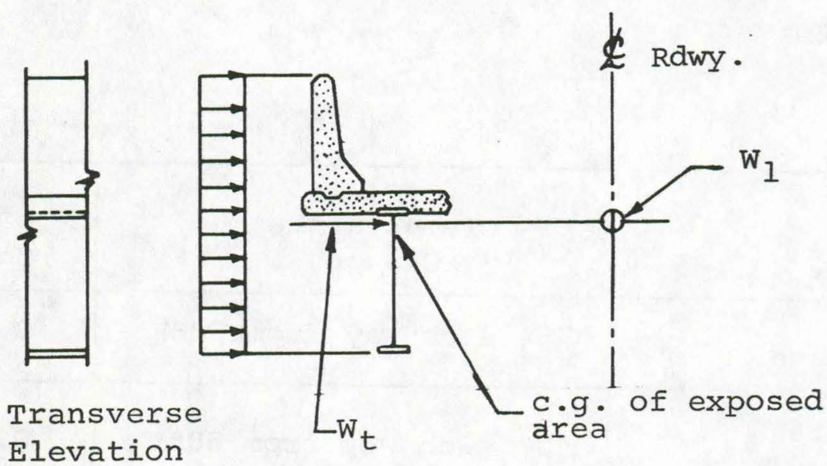
(c) The reference to friction in Article 1.2.13 is interpreted to be a force due to temperature change as is discussed on pages 7 and 8, and is not to be included as part of the LF.

(6) Wind Loads Art. 1.2.14

(a) Wind load on superstructure (W)

W reaction, transverse and longitudinal, to each pier except as modified in (8) and (9) for the piers of multiple span continuous beam bridge = (Summation of area per linear ft. of barrier slab and exterior beam as seen in the transverse elevation) x (Average span length of two spansadjacent to the pier under consideration) x (Unit W as stated in Art. 1.2.14 (B)).

The transverse wind (W_t) and the longitudinal wind (W_l) shall apply simultaneously at c.g. of the exposed area as seen in the transverse elevation, except as modified in (8), (9), and (10).



(b) Wind load on moving live load (WL)

WL reaction, transverse and longitudinal, to each pier except as modified in (8) and (9) for the piers of multiple span continuous beam bridge = until WL as stated in Art. 1.2.14 (B)1 x (Average length of two spans adjacent to the pier under consideration).

The transverse WL_t and longitudinal WL_l shall apply simultaneously at 6ft. above roadway, except as modified in (8), (9) and (10).

(c) Wind Overturning Forces (WOF)

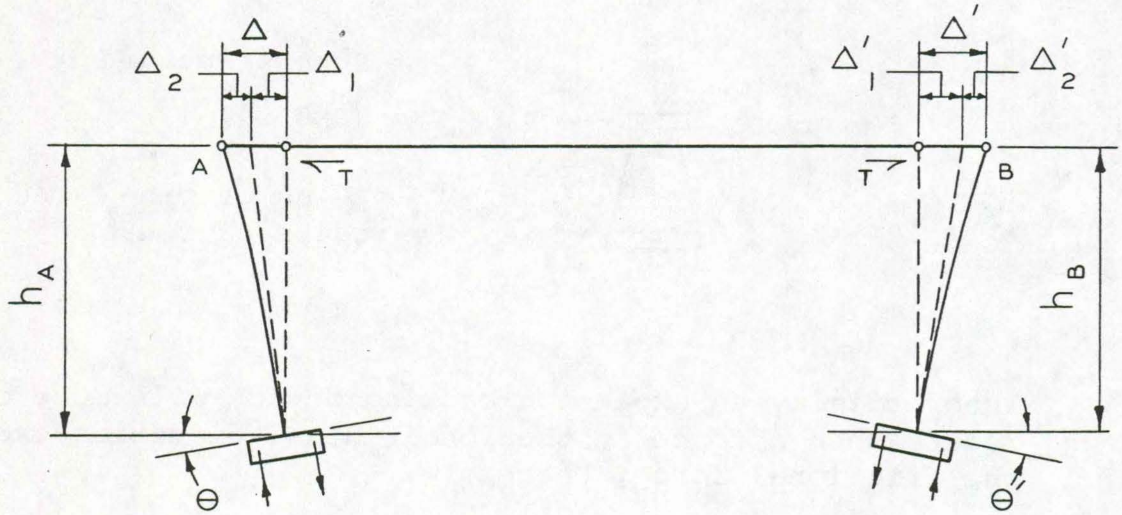
It is not necessary to investigate the effect of wind overturning forces on pedestal (rigid frame bent) piers. It shall be applied to the design of T-pier and diaphragm piers.

(7) Thermal Force (T) Art. 1.2.15

(a) For design purpose, the following ranges of temperature and thermal coefficients shall be used to calculate the deformation of structures due to temperature change:

	Design Temperature Change	Thermal Coefficient
Steel Structure	75° each way from 50°F	0.000,006,5
Concrete Structure	50° each way from 50°F	0.000,006

(b) In case the superstructure is fixed at its ends by the piers, both the fixed piers will be subject to an external thermal force due to temperature change in the superstructure. If the fixed piers in this case rest on pilings as shown below, the effect of pier shaft deflection as well as the footing rotation shall be taken into consideration in determining the magnitude of the thermal force.



$\Delta + \Delta'$ = total change in span length AB due to temperature change.

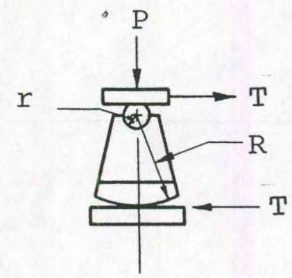
$\Delta_1 = \Theta \times h_A$ & $\Delta'_1 = \Theta' \times h_B$ = displacement at pier top due to footing rotation caused by thermal force T.

Δ_2 & Δ'_2 = deflection at pier top due to bending of pier shaft by thermal force T.

See 13 (b) for effective piling length to be used in the calculation of footing rotation.

(c) The direct load used in calculating the thermal force at the expansion bearings shall be the superstructure dead load reaction. For design purposes, a frictional coefficient of <u>0.25</u> shall be used for steel bearing on steel, and a coefficient of <u>0.10</u> be used for steel bearing on self-lubricating bronze plate.

In calculating the thermal force under steel bearing rockers, the mechanical property of the rocker shall be taken into consideration as below:



P = Superstructure DL

$T = (P \times 0.25)\left(\frac{r}{R}\right)$

r= radius of pin
R= radius of rocker

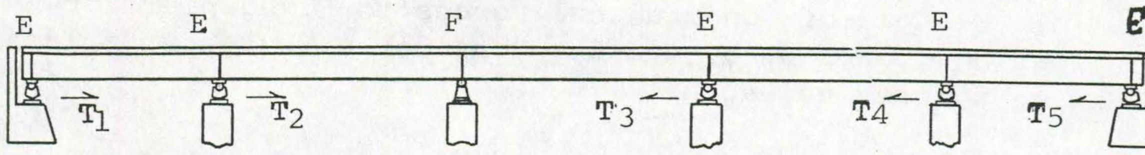
Theoretically an expansion bearing device will move or rotate when a force in the longitudinal direction exceeds the frictional force, therefore:

(1) The thermal force is the external thermal force, T, caused by temperature change in the superstructure, which is transmitted through an expansion bearing device, and also

(2) The thermal force is the limit of all forces in the longitudinal direction that can be transmitted through an expansion bearing device.

(d) Pedestal piers shall be designed to include the internal thermal force and shrinkage forces within the frame members.

(e) In a continuous superstructure resting on bearing devices, the fixed pier or piers shall also resist the unbalanced thermal force from expansion bearings.



Unbalanced thermal Force at Fixed Pier

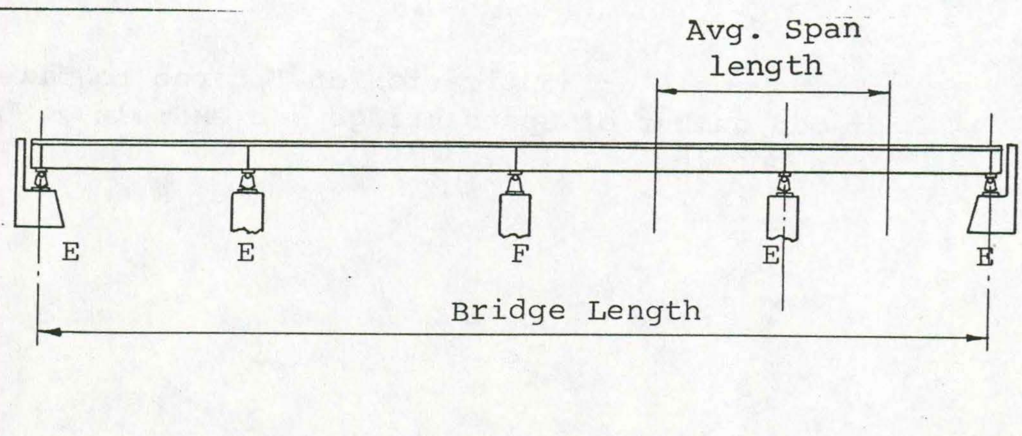
$$= (T_1 + T_2) - (T_3 + T_4 + T_5)$$

In all cases, even when the unbalanced thermal force is zero, the fixed pier shall be designed with a total longitudinal force of not less than the thermal force applied to the expansion piers. See 8(b) for example.

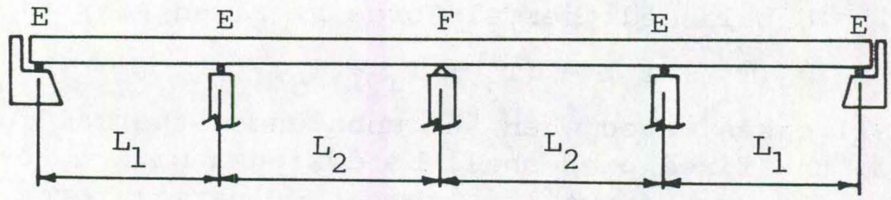
Bridges utilizing neoprene bearings (laminated or unlaminated) are assumed to have all fixed piers, and the thermal force applied to each pier shall be equal.

(8) Application of W, WL, LF and T to 0° Skew Piers of A Multiple Span Continuous Beam Bridge

(a) Because the thermal force is the limit of any longitudinal force that can be transmitted to the pier through a rocker as stated in (7c), the distribution of longitudinal W_L, WL_L, LF and T to both expansion and fixed piers of a multiple span continuous beam bridge is modified in AASHTO Loading Groups II, III, V, VI and IX as shown in Tables I to IV. In the Tables I to X the term "average span length" is defined as the average length of two spans adjacent to the pier under consideration and the term "bridge length" is defined as span length between centerline to centerline abutment bearings. When I.D.O.T. standard rockers and fixed shoes are used, it is assumed that all longitudinal forces are transmitted through rockers at bridge seat level & through fixed shoes at one foot (1'-0) above bridge seat.



(b) Example of Longitudinal Forces to Fixed pier for loading groups 4, 5, and 6. Consider a four span bridge with symmetrical spans.



(1) Group 4. Only longitudinal force to fixed pier is T. T theoretically is 0, but in accordance with (7)(e) it shall be applied to the fixed pier in the same magnitude as applied to the expansion piers.

(2) Group 5. Longitudinal forces applied to fixed pier are W_L (bridge length) and T. If W_L (bridge length) is greater in magnitude than the T applied to expansion piers then T applied to fixed pier is 0. If W_L is smaller than T to exp. pier then T to fixed pier is the difference between T (expansion) and W_L.

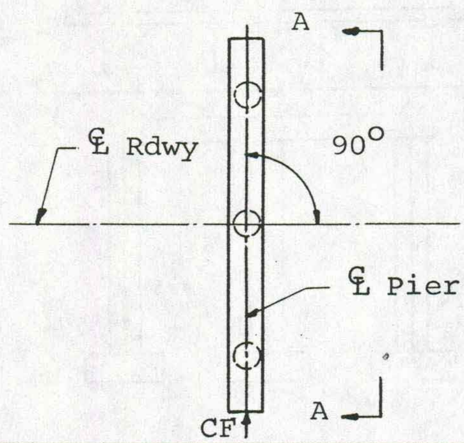
(3) Group 6. Longitudinal forces applied to fixed pier are W_L, WL_L, and LF (all based on bridge length) and T. As in group 5 if the summation of W_L, WL_L, and LF are greater than T applied to expansion pier then T to fixed pier is 0. If not then T to fixed pier is difference between T (expansion) and $W_L + WL_L$ + LF.

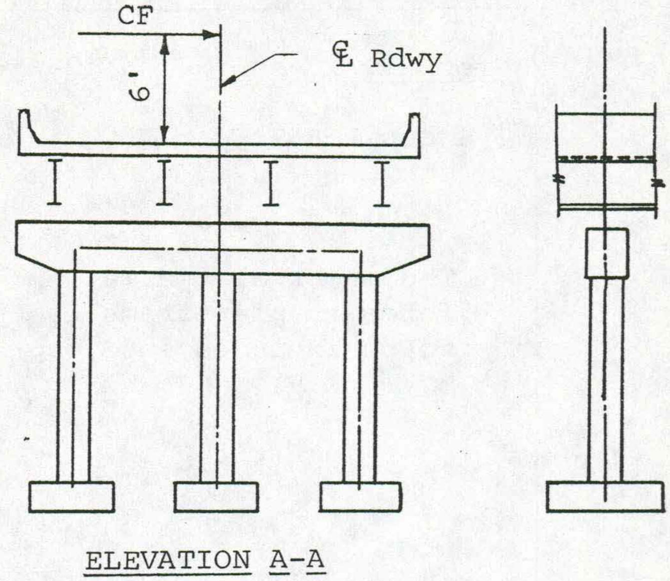
(c) For example of application of T force to fixed pier of odd number of spans bridge see example at back of this booklet.

TABLE I — EXPANSION PIERS — 0° SKEW

Group I 100%

D+L+I+E+B+SF+CF





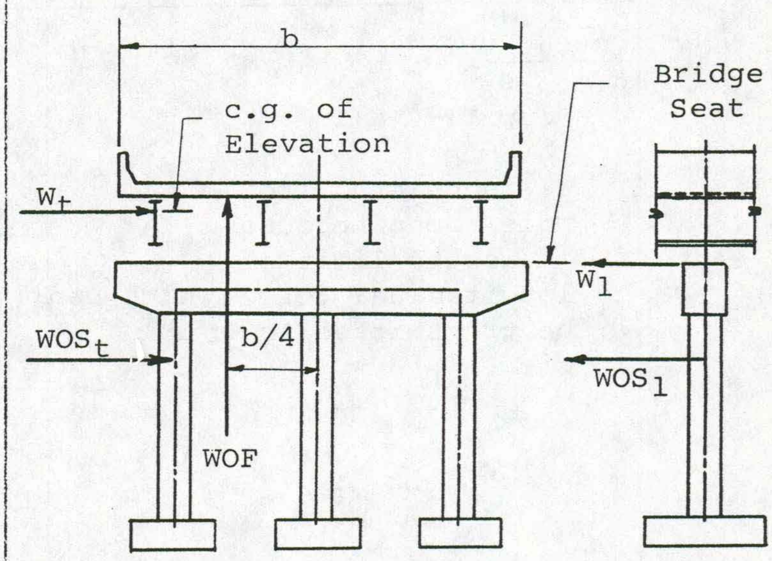
Group II 125%

D+E+B+SF+W

W includes:

(1) WOS (Wind on substr.)
(2) W_t on avg. span length
(3) W_l on avg. span length
(4) WOF (Wind overturning force) ①

See Note ① on P. 14.



Group III 125%

Group I+LF+.3W+WL

LF, .3W & WL on avg. span length

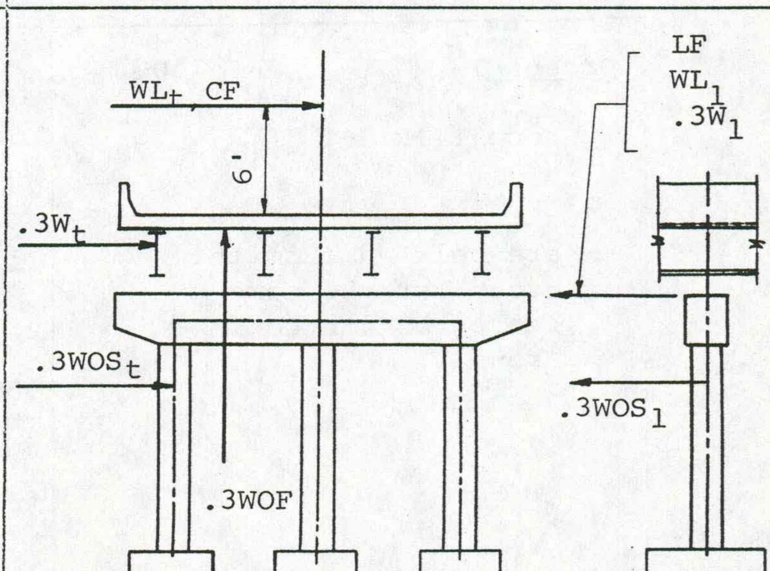


TABLE II — EXPANSION PIERS — 0° SKEW

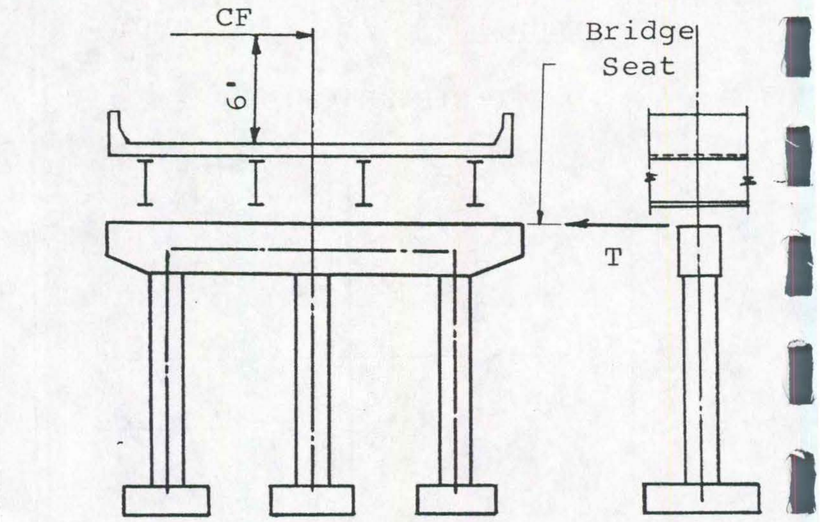
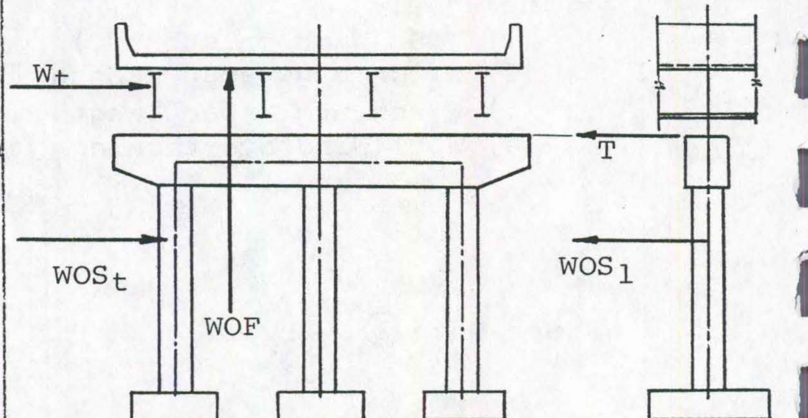
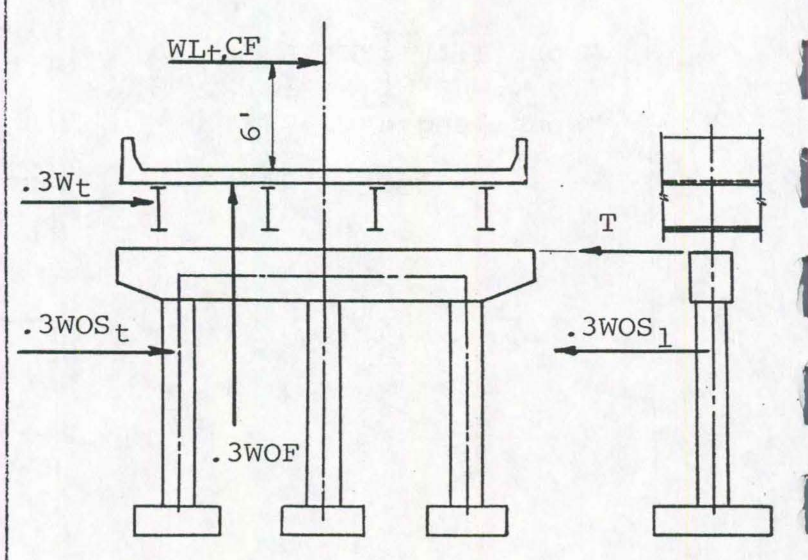
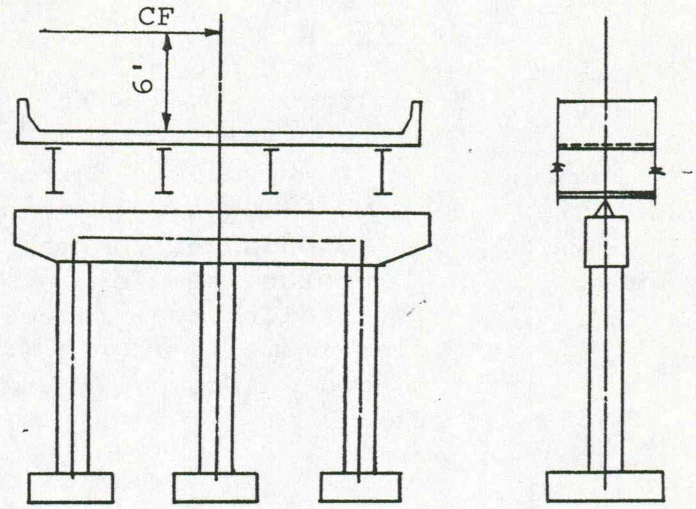
Group IV — 125% Group I+R+S+T The internal thermal force in pier frame is to be included in Group IV, V & VI	
Group V — 140% Group II+R+S+T W_l is deleted from this group because T is the limit of longitudinal force which can be transmitted to this pier.	
Group VI — 140% Group III+R+S+T LF, WL_l, & $.3W_l$ are deleted from this group for the same reason as stated in Group V.	

TABLE III — FIXED PIER — 0^{o} SKEW

Group I 100%

D+L+I+E+B+SF +CF



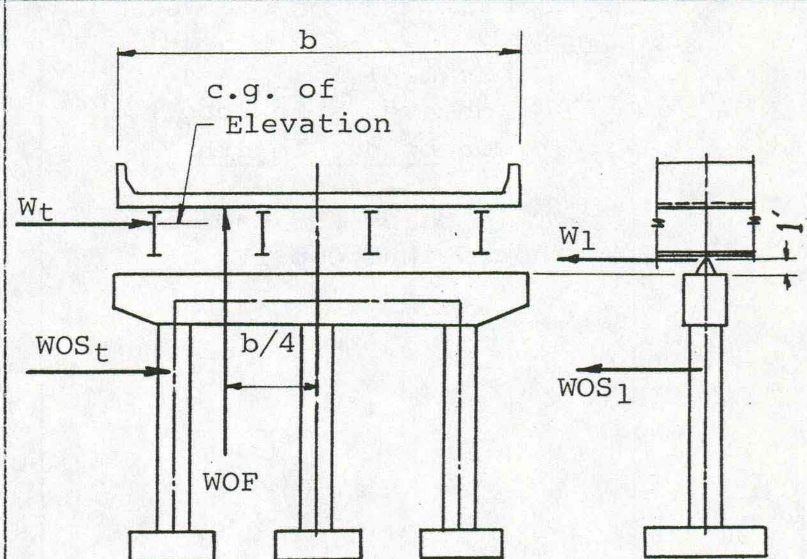
Group II 125%

D+E+B+SF+W

W includes:

(1) WOS
(2) W_t on avg. span length
(3) W_l on avg. span length
(4) WOF (Wind overturning force) ①

See Note ① on Pg. 14.



Group III 125%

Group I+LF+.3W+WL
LF, .3W & WL on avg. span length.

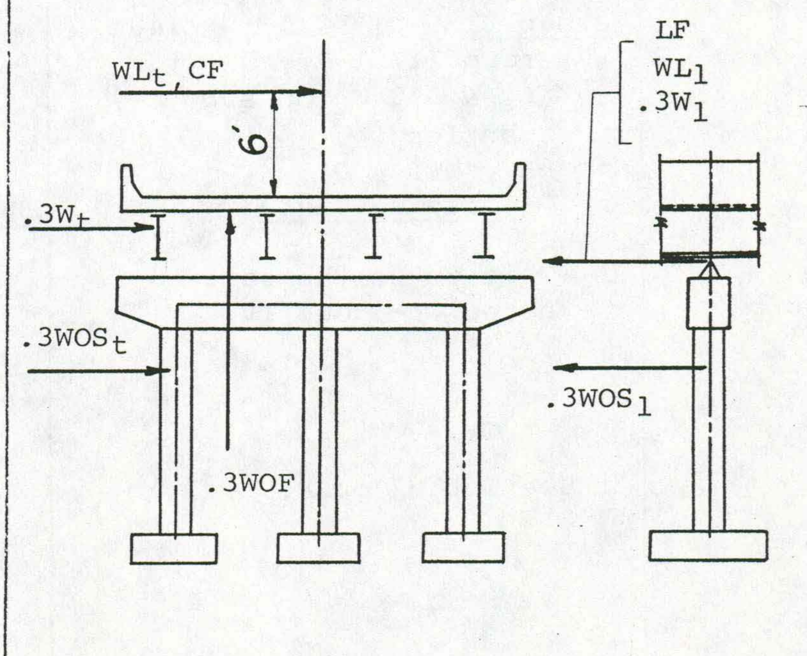


TABLE IV — FIXED PIER — 0° SKEW

Group IV — 125%

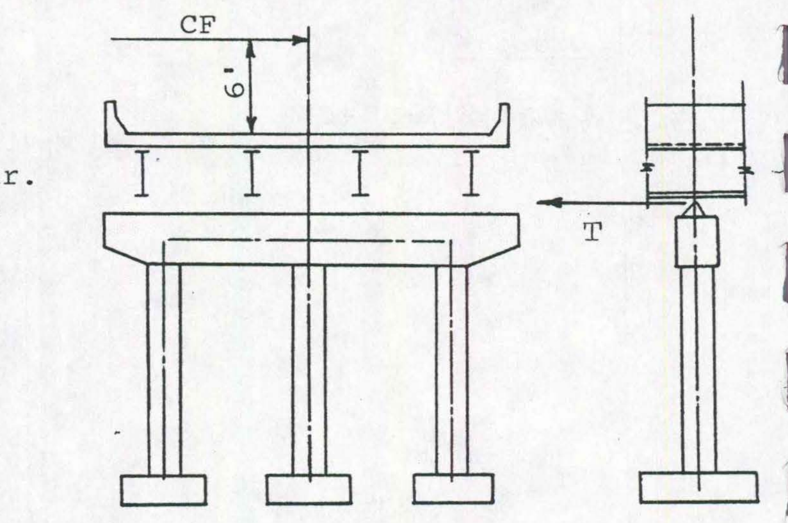
Group I + R + S + T

T includes:

(1) thermal force due to temp. change in superstr. between two fixed piers

(2) unbalanced thermal force (see 7e)

Internal thermal force in pier frame is *also to be in*cluded in Group IV, V & VI



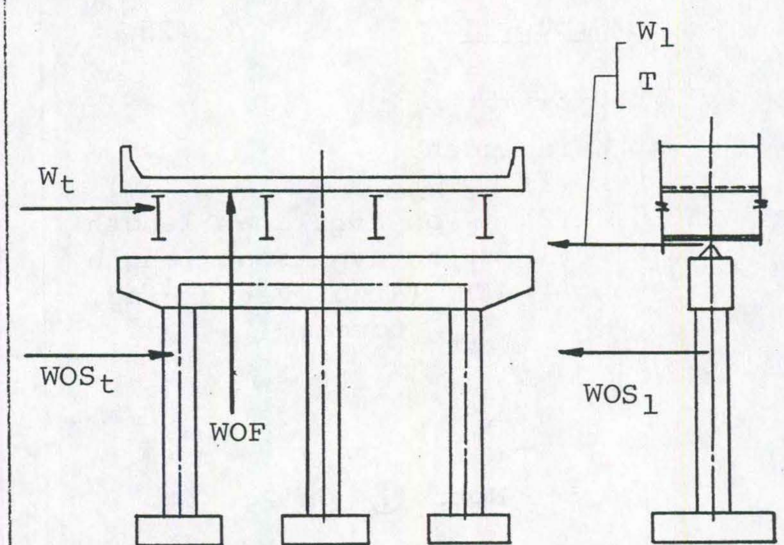
Group V — 140%

Group II + R + S + T

W_t on ave. span length.

W_l on bridge length ②

T is to be the same as noted in Group IV.



Group VI — 140%

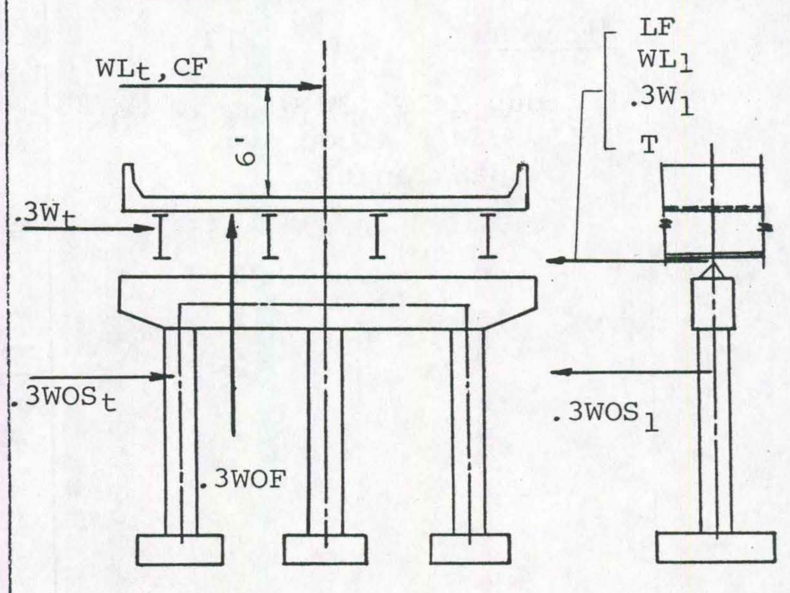
Group III + R + S + T

WL_t & 0.3 W_t on avg. span length.

LF_l, WL_l & 0.3 W_l on bridge length ②

T is to be the same as noted in Group IV

See Note ② on P. 14.

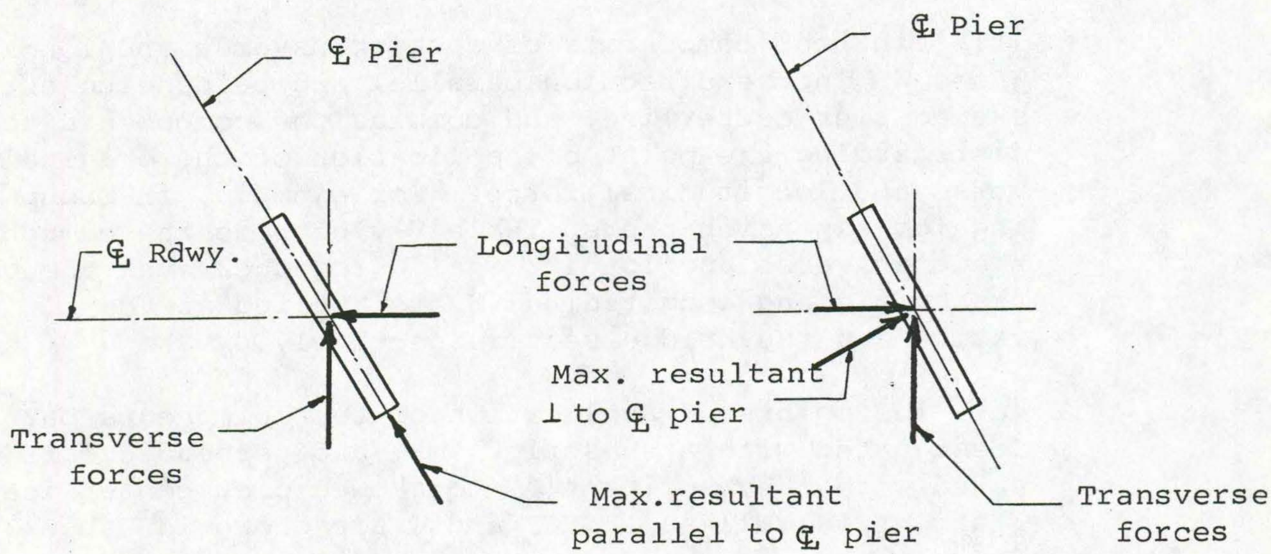


Note ① As stated in 6(c), it is not necessary to investigate the effect of WOF on pedestal piers. The WOF shown in Tables I to X is for the design of other types of piers.

Note ② In Group V and VI loading combinations, it is assumed that all forces in the longitudinal direction are applied simultaneously with the thermal force T. It is further assumed that they cannot be transferred to the expansion pier through the rockers because of rocker rotation due to the thermal force. Therefore, all longitudinal forces, other than the thermal force, are resisted by the fixed pier or piers and are calculated on the basis of bridge length.

(9) <u>Application of W, WL, LF and T to Skewed Piers of A Multiple Span Continuous Beam Bridge</u>

When designing skewed piers, the direction of longitudinal forces (W_l, WL_l, LF and T) shall be arranged in such a way so that their components in conjunction with the corresponding components of transverse force W_t and WL_t will produce maximum resultant parallel or perpendicular to skewed pier centerline as illustrated below.



For the design of 0^o skew piers of a multiple span continuous beam bridge, it can be seen in Tables I through IV that the transverse and longitudinal W and WL are not applied at the same level (such as Group II for expansion piers), same time (such as Group V for expansion piers), and in some cases, their magnitudes are determined on the basis of different length along the bridge (such as Group V for fixed piers). When the piers are skewed, the above-mentioned transverse and longtudinal W and WL will have to be resolved into components parallel and perpendicular to the pier centerline thus resulting in rather complicated loading patterns. For instance, in Group II loading for expansion pier (Table I), the component of transverse W parallel to the skewed pier centerline will apply at the c.g. of elevation while the component of the longitudinal W in the same direction will apply at the bridge seat level. In order to simplify the loading pattern in the design of skewed piers, the following emperical rules are used with regard to transverse and longitudinal W and WL:

(a) Calculate the transverse and longitudinal W and WL in the same manner as stated in Tables I through IV for 0^o skew piers.

(b) Find the components of each transverse and longitudinal W and WL in the direction parallel and perpendicular to skewed pier centerline, and combine the components together disregarding the point of application of the original transverse and longitudinal force. For example, in Group II loading for expansion piers (Table V), combine the components of the transverse and longitudinal W together even though the transverse and longitudinal W are applied at the c.g. of elevation and the bridge seat respectively.

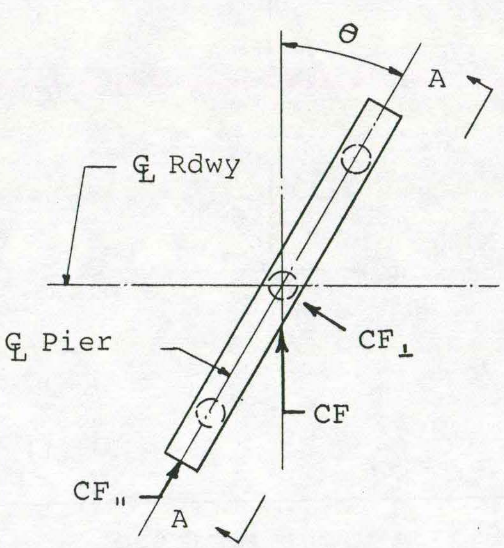
(c) The points of application of the components parallel (designated with a subscript "$\parallel$") and perpendicular (designated with a subscript "$\perp$") to skewed pier centerline of the W and WL as well as LF, CF and T are shown in Table V through X.

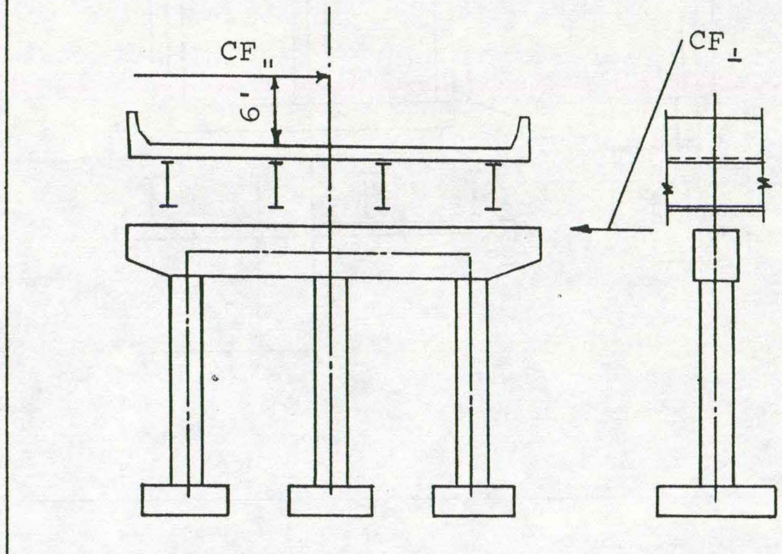
Table V

SKEWED EXPANSION PIER

Group I 100%

D+L+I+E+B+SF+CF



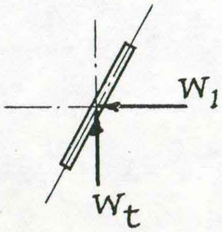


ELEVATION A-A

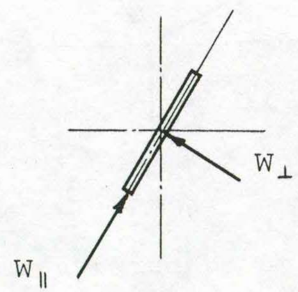
Group II 125%

D+E+B+SF+W

(1) Calculate longit. and trasv. wind on superstructure (W) based on avg. span length.



(2) Calculate the components disregarding pt. of applications.



(3) Apply W∥ & W⊥ at locations shown at right.

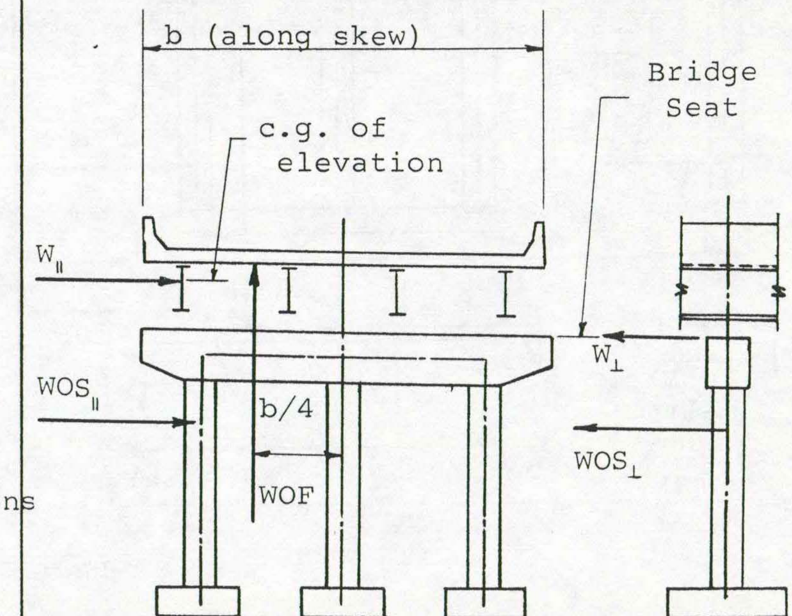
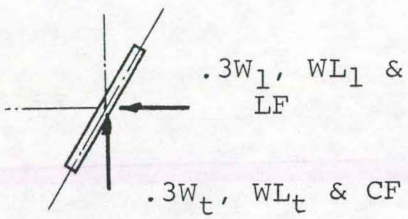


TABLE VI SKEWED EXPANSION PIER

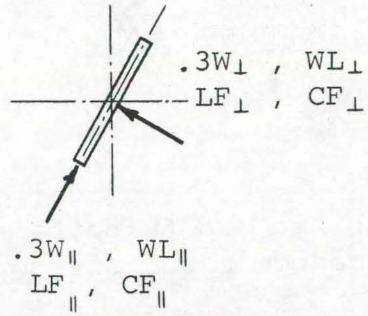
Group III 125%

Group I + LF + .3W + WL

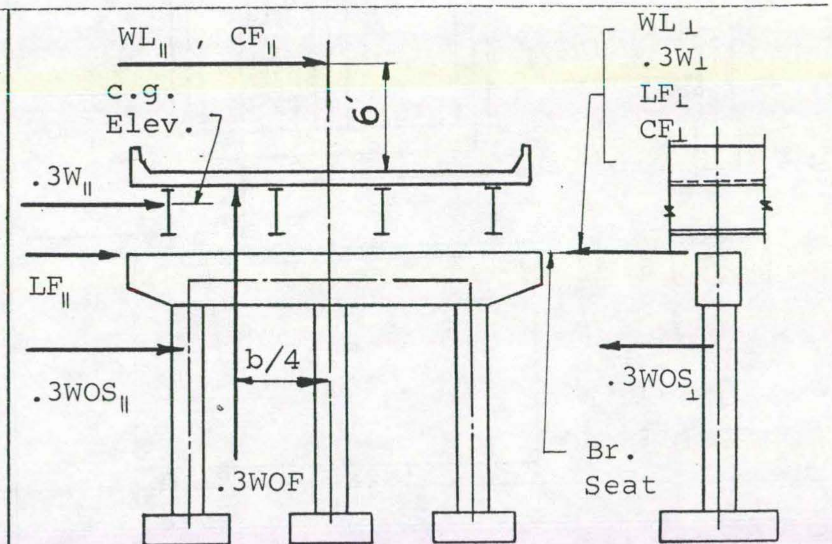
(1) Calculate longit. and transv. .3W and WL, & LF on avg. span length.



(2) Calculate the components disregarding pt of applications.



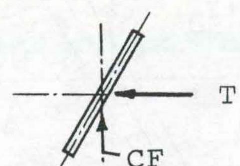
(3) Apply the components at locations shown at right.



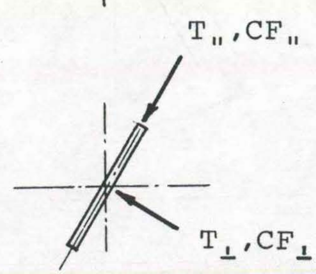
Group IV 125%

Group I + R + S + T

(1)



(2)



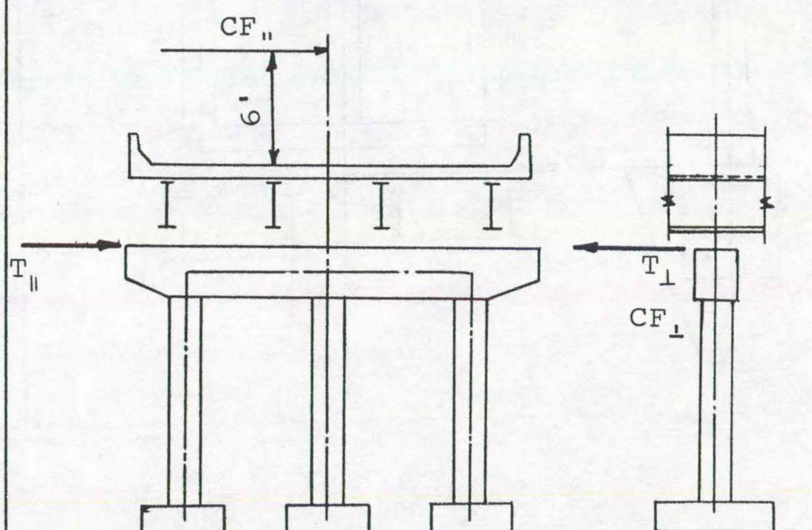


TABLE VII

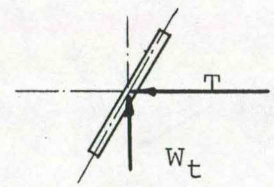
SKEWED EXPANSION PIER

Group V 140%

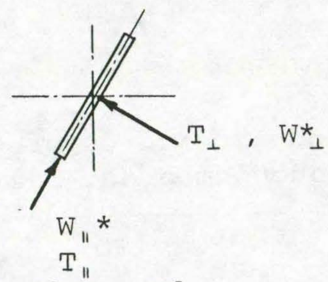
Group II + R + S + T

W_l is deleted from this group because T is the limit of longitudinal force which can be transmitted to this pier.

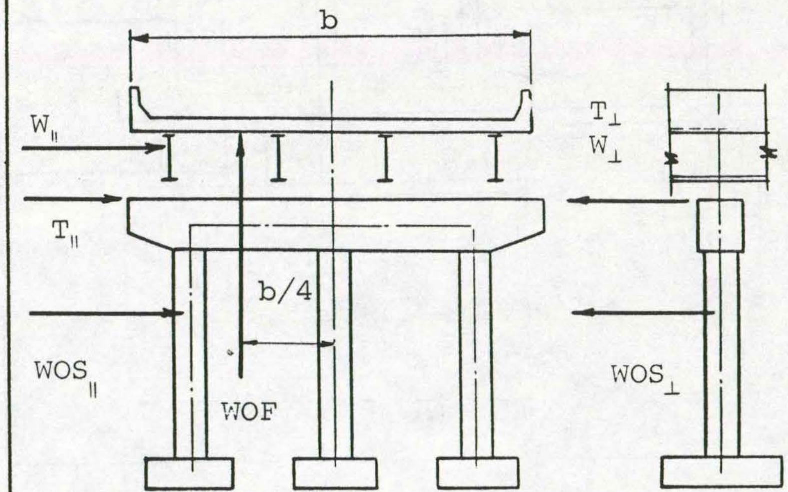
(1)



(2)



*Component of W_t only

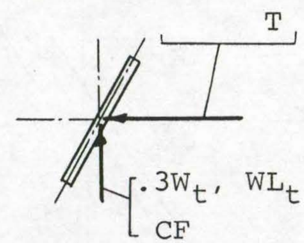


Group VI 140%

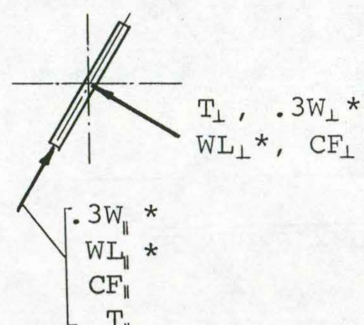
Group III + R + S + T

$.3W_l$, WL_l & LF are deleted in this group for the same reason as stated in Group V.

(1)



(2)



* Component of $.3W_t$ & WL_t only

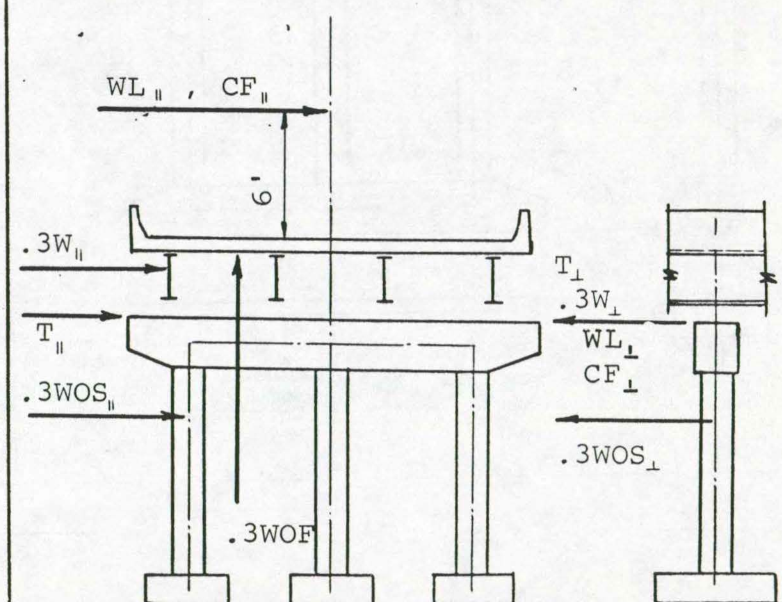
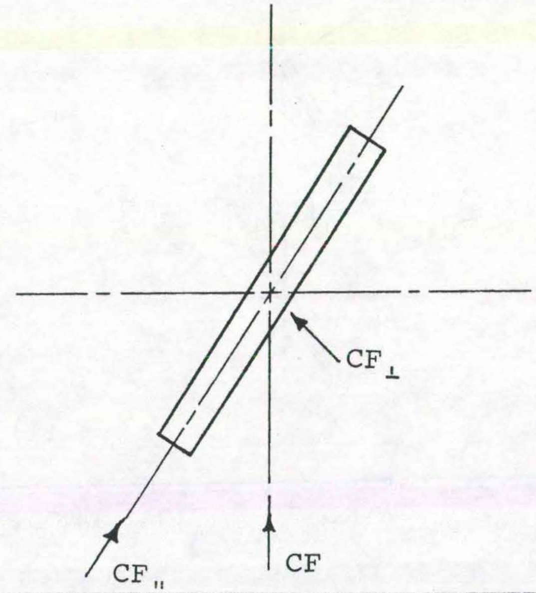


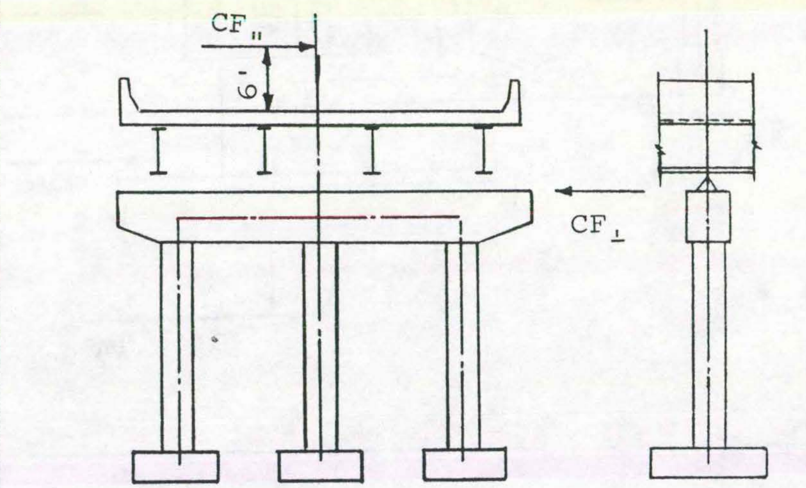
TABLE VIII

SKEWED FIXED PIER

Group I 100%

D+L+I+E+B+SF+CF

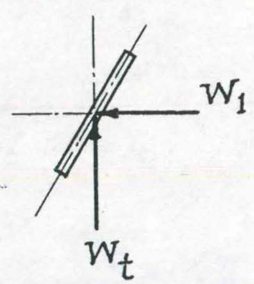




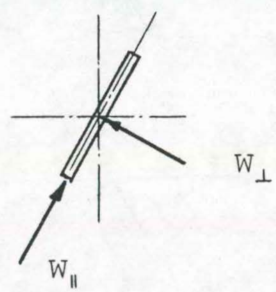
Group II 125%

D+E+B+SF+W

(1) Calculate longit. and transv. W based on avg. span length.



(2) Calculate the components disregarding pt. of application.



(3) Apply $W_{\parallel}$ & $W_{\perp}$ at locations shown at right.

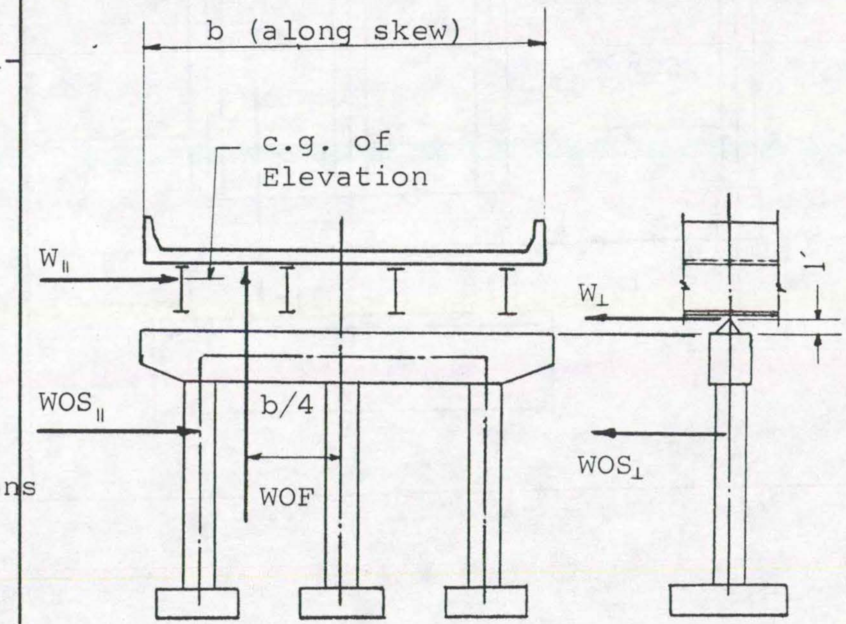
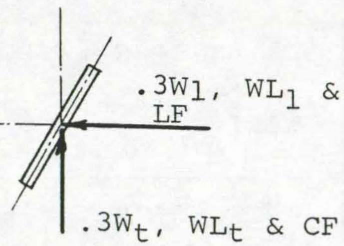


TABLE IX — SKEWED FIXED PIER

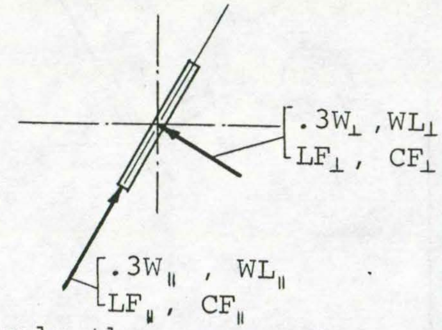
Group III — 125%

Group I+LF+.3W+WL

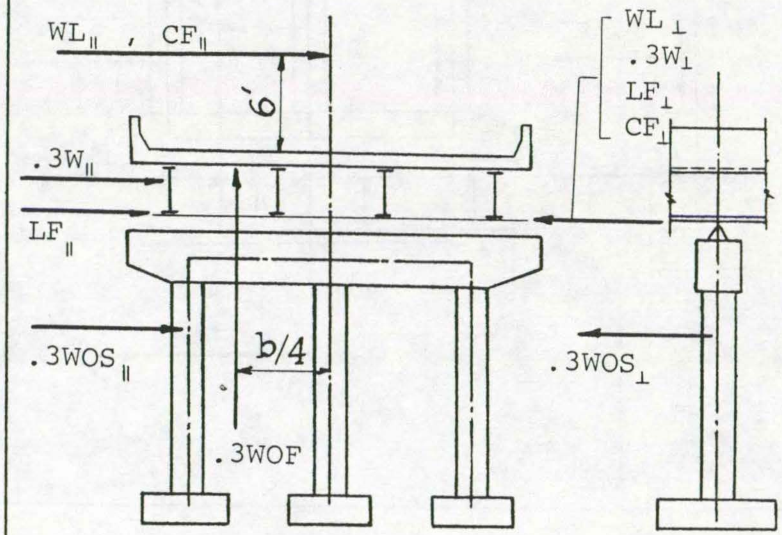
(1) Calculate longit. and transv. .3W and WL, & LF on avg. span length.



(2) Calculate the components disregarding pt. of applications.

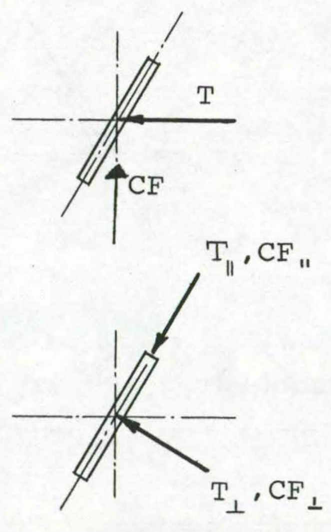


(3) Apply the components at locations shown at right.



Group IV — 125%

Group I+R+S+T



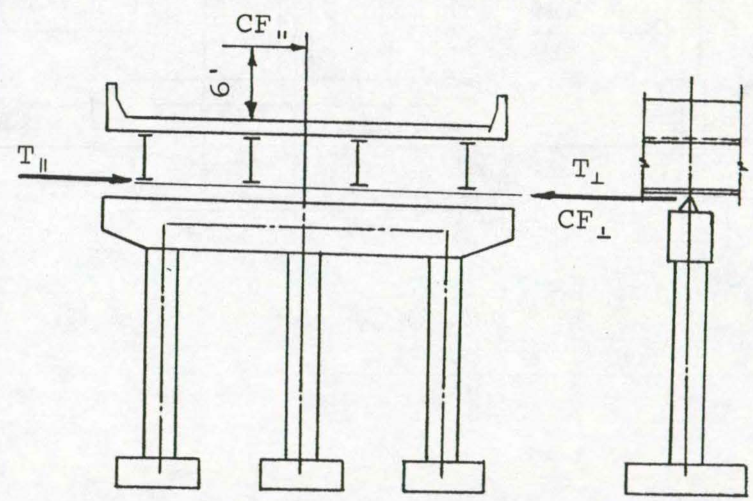
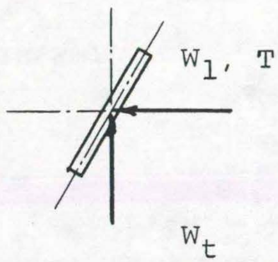


TABLE X SKEWED FIXED PIER

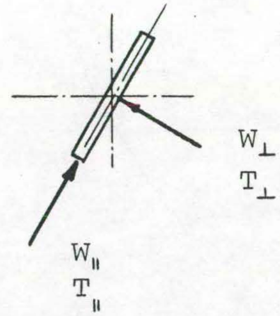
Group V 140%

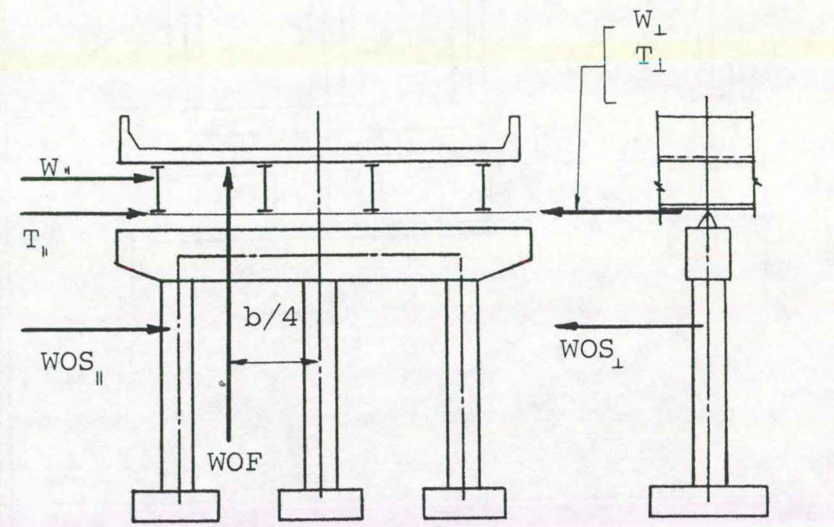
Group II + T

(1) Calculate transv. W_t on avg. span length and longit. W_l on bridge length ②



(2) Calculate the components

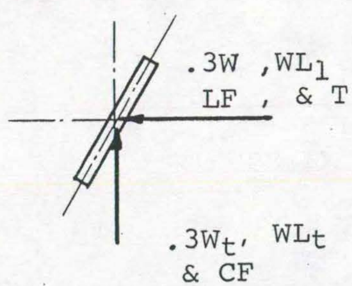




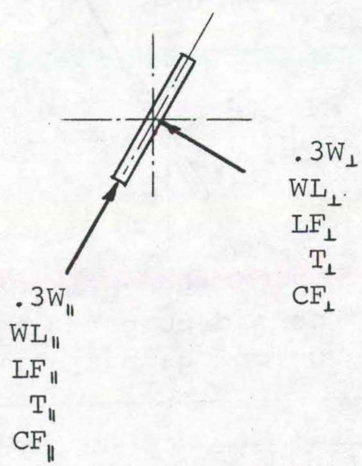
Group VI 140%

Group III + R + S + T

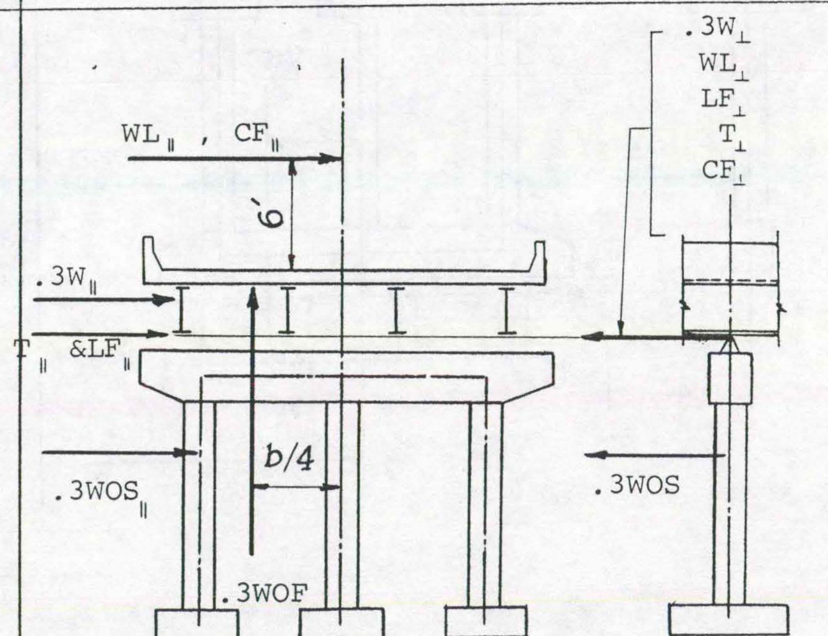
(1) Calculate transv. $0.3W_t$ & WL_t on avg. span length. Calculate longit. $0.3W_t$, WL_l & LF on bridge length ②.



(2) Calculate the components

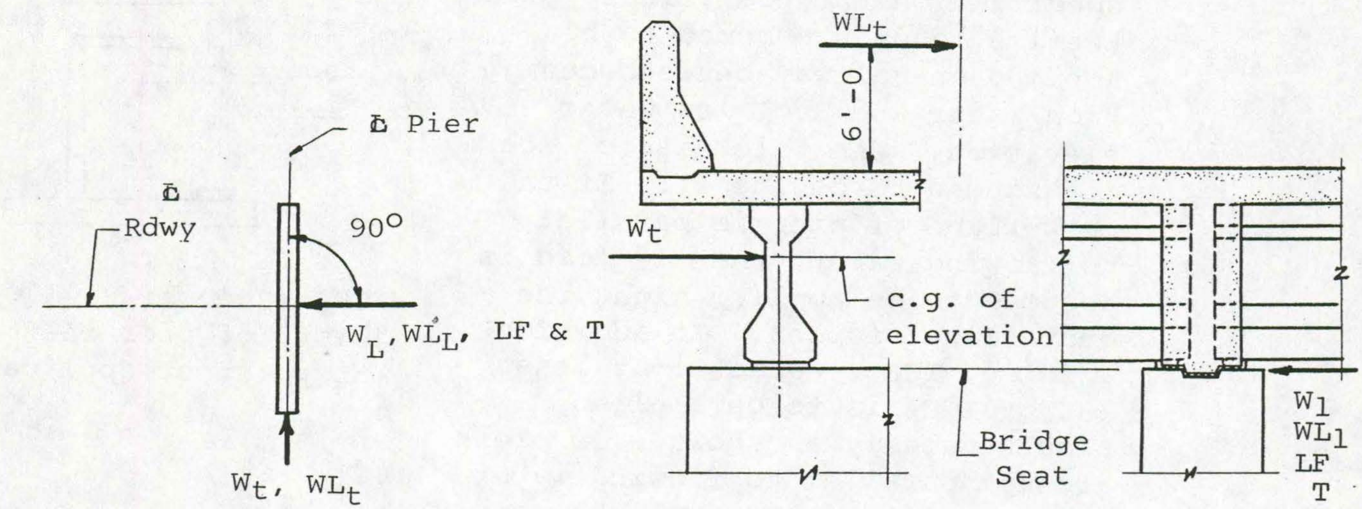


See Note ② on P. 14



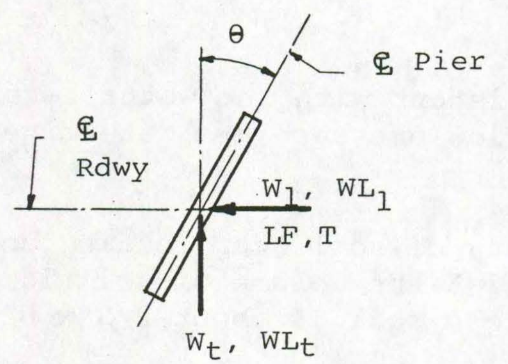
(10) Application of W, WL, LF and T to Piers of A Prestressed Concrete Beam Bridge

(a) 0° Skew pier:

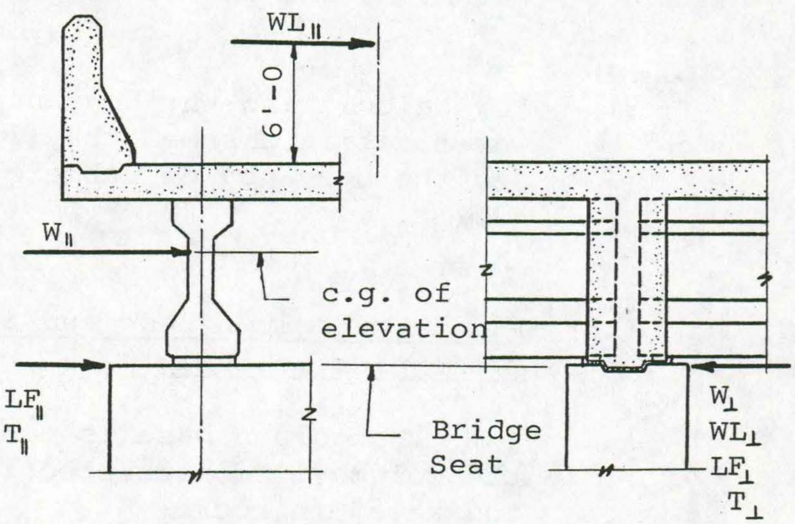


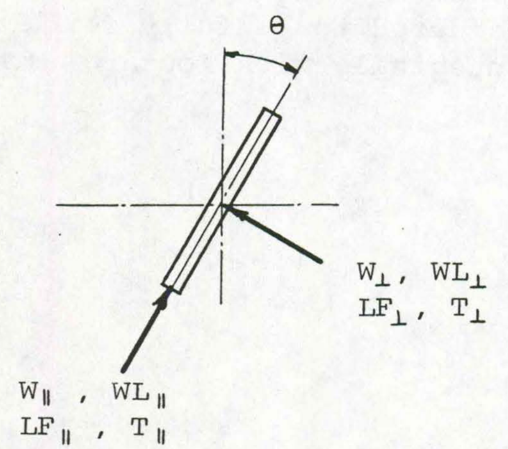
W and WL are based on average span length.

(b) Skewed pier:



W and WL are based on average span length.

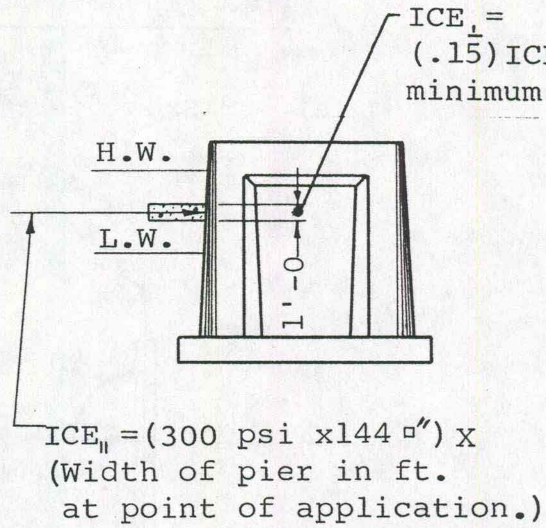


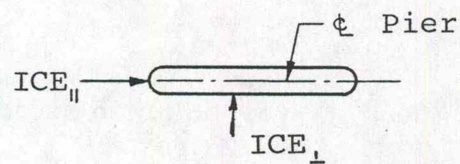


Calculate $W_{\perp}$, $W_{\parallel}$, $WL_{\perp}$ and $WL_{\parallel}$ according to (9b)

(11) Ice Load (ICE) Art. 1.2.17.

Unless the elevation of ice is specified on the preliminary plan, the ice load is assumed to be applied at halfway between design high water and avg. low water elevation. The thickness of ice is assumed to be one ft. If the centerline of pier is parallel to flowing water, the ice load is assumed to be applied along the centerline of pier. In addition a force, $ICE_{\perp}$, equal to at least 15% of $ICE_{\parallel}$ is to be applied simultaneously as shown. If piers are not parallel to flowing water the total ICE force shall be computed as in 1.2.17 (B)(1) and resolved into vectors.





(12) Bouyancy Art. 1.2.18.

The bouyancy shall be consistent with the water level from which the ice and stream flow pressure are determined.

For calculating the bouyancy of soil over footing under water, it is assumed that 1/3 the volume of soil is void, and the bouyancy of submerged soil is about 2/3 x 62.4 = 42#/ft.^{3}.

(13) Fixity of column base and transverse forces in rigid frame pier analysis

(a) The column base is considered to be fully fixed when the column is constructed integrally with footings founded and keyed in rock.

(b) The column base is considered partially fixed when the column is constructed integrally with footings founded on pilings. The degree of fixity depends upon the moment inertia of piling group and pile length. The footing fixity is defined as:

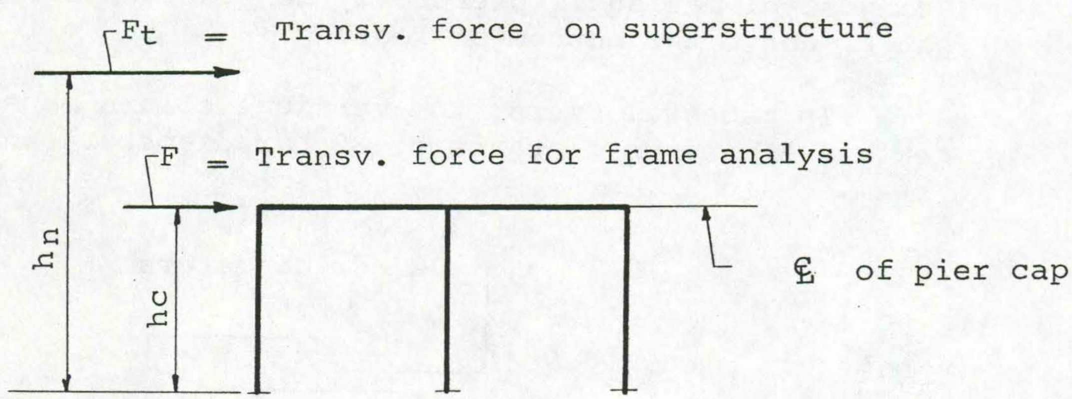
$$\frac{J}{J + K_c}$$

J = moment required to rotate footing one radian.

K_c = stiffness factor of column constructed integrally with the footing.

The pile length used to determine the footing rotation is assumed to be 50% and 75% of the pile length in ground for timber and steel piling respectively.

(c) In transferring the transverse forces applied on the superstructure to the pier cap, the following criteria is used.



$$F = F_t \cdot \frac{h_n}{h_c}$$

(14) Miscellaneous Pier Details

(a) The distance from center of exterior piles to the nearest edge of footing shall be not less than 1'-6.

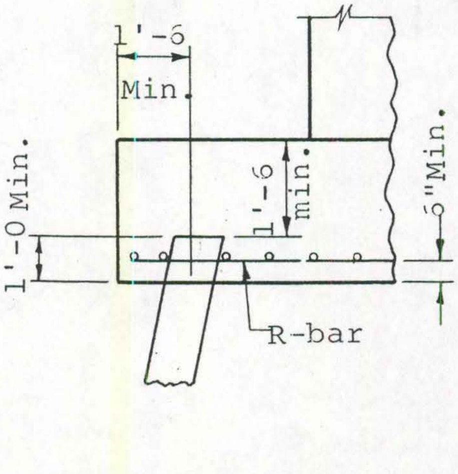
(b) The top of piling shall project not less than 1'-0 into the concrete footing.

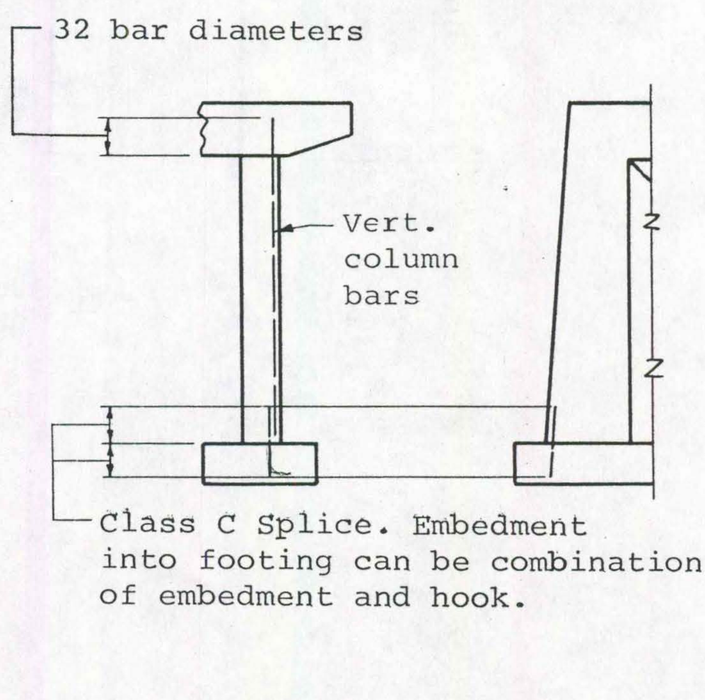
(c) If the top of piling protrudes above the bottom reinforcing mat, the remaining depth of footing above the top of piling shall not be less than 1'-6.

(d) The dowel bars shall extend into the column or pier shaft and footing a minimum distance of a Class C Splice for the appropriate bar size.

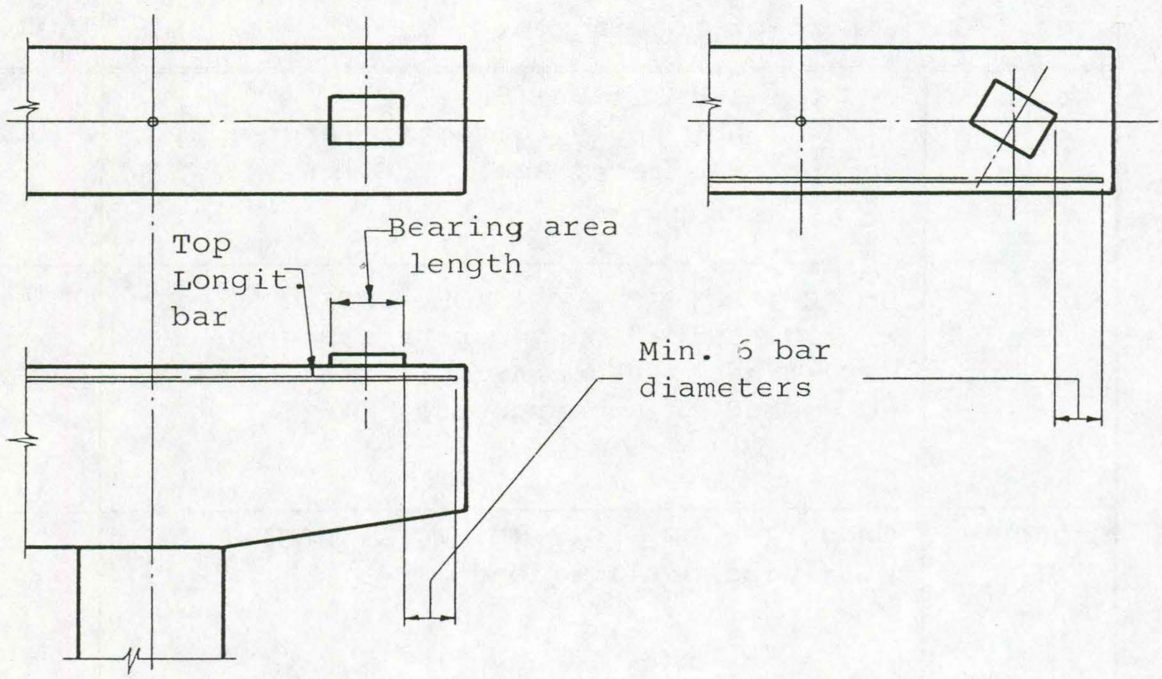
(e) For timber piling footings, the reinforcing steel mat shall be placed 1½" clear above top of the piling. For steel piling footings with close piling spacings, such as pedestal pier column footings, the reinforcing steel mat shall preferably be placed on top of the piling. For steel piling footings with wider piling spacing, such as diaphragm pier footings, the reinforcing steel mat shall preferably be placed between piling at 6" above the bottom of footing.

(f) In pedestal piers, the vertical column bars shall extend a minimum distance of 32 bar diameters into the cap beam.





(g) The minimum anchorage length of top longitudinal reinforcing bars beyond bearing area in an overhanging pier cap shall be 6 bar diameters, but the required development length, l_d, of the top longitudinal bars must be satisfied.



(15) <u>Uplift in friction piling</u>.

The allowable bond stress between the tremie concrete and piles to resist hydrostatic uplift pressure, shall be 10 psi.

For piles embedded in concrete footing, the allowable bond stress between concrete and piles to resist intermittent uplift load shall be 15 psi.

No uplift is allowed in footing piles under DL and LL.

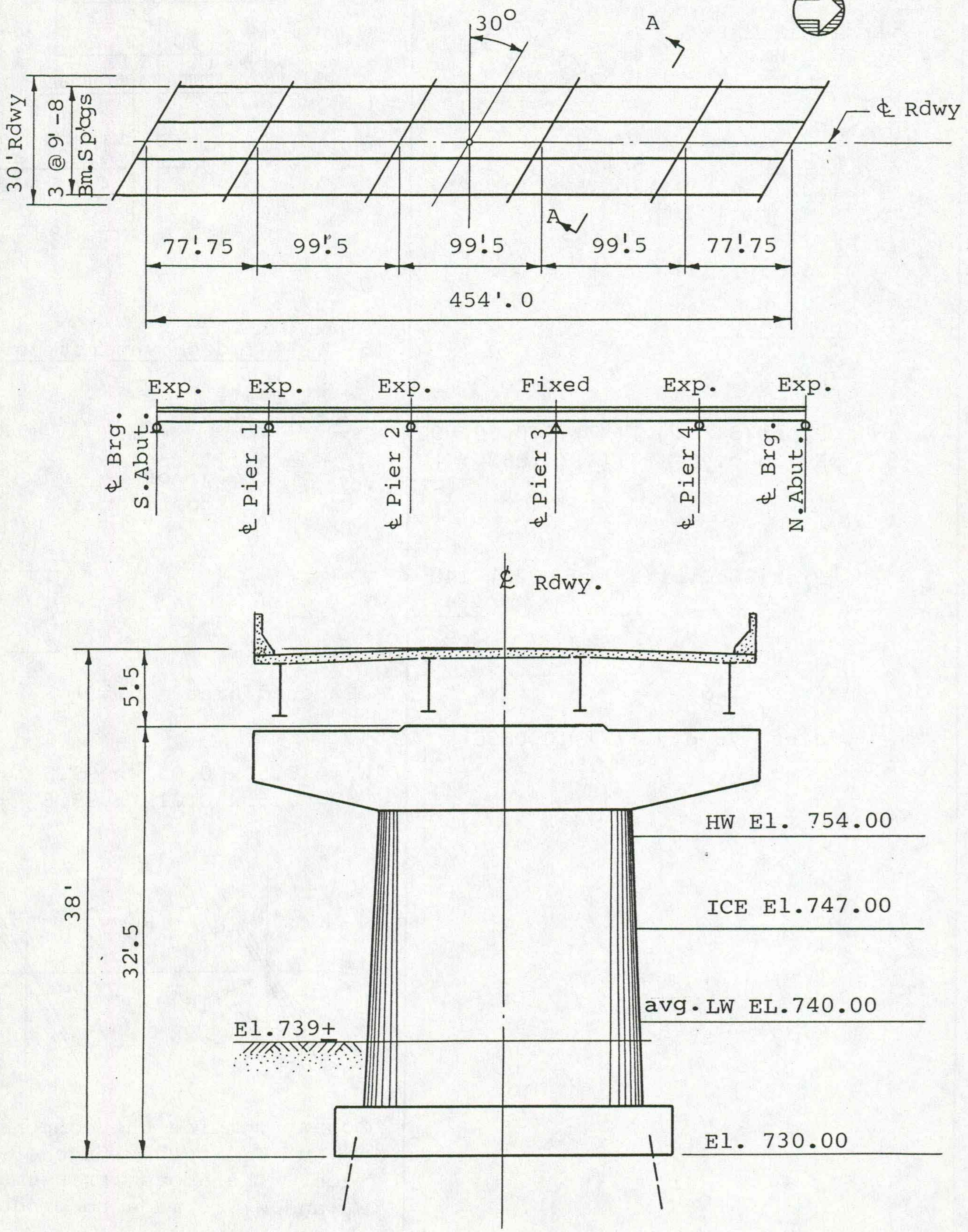
(16) Combination of Loads and Forces on Piers Art.1.2.22

	Group of Loadings	% of Unit Stress
Group I	D + L + I + E + B + SF + CF I shall not apply to footing and piling, except pile bent. See (3).	100%
Group II	D + E + B + SF + W + WOF + WOS W = Wind load on superstructure WOF= Wind overturning force WOS= Wind on substructure	125%
Group III	Group I + LF + 0.3 (W+ WOF + WOS) + WL WL = Wind on live load	125%
Group IV	Group I + R + S + T *	125%
Group V	Group II + R + S + T*	140%
Group VI	Group III + R + S + T *	140%
Group VIII	Group I + ICE	140%
Group IX	Group II + ICE	150%

* T shall include (1) thermal force at expansion bearing or unbalanced force at fixed piers due to temperature change in the superstructure, and (2) internal force in pier frame due to temperature change plus shrinkage.

Example

Pier Piling Design for A 30^{o} Skew 454'x 30' Five Span Continuous I-Beam Bridge



SECTION A - A

Pier #2 - Expansion Pier

Loadings

(1) Super D

Ext.Bm. 2 x 134.^{k}8 = 269.^{k}6

Int.Bm. 2 x 151.^{k}7 = 303.^{k}4

573.^{k}0

(2) Pier D   k/ft.^{3}

(103 c.y.)(27)(0.15)= 417.^{k}2

(3) E   k/ft.^{3}

(36 c.y.)(27)(0.1) = 98.^{k}0

(4) Bouyancy

(a) LW

Pier (54 c.y.)(27)(0.0625)= 90^k

E (36 c.y.)(27)$\left(\frac{2}{3}\right)$(0.0625)= 41^k

131^k

(b) HW

Pier(88c.y.)(27)(0.0625)= 148^k

E = 41

189^k

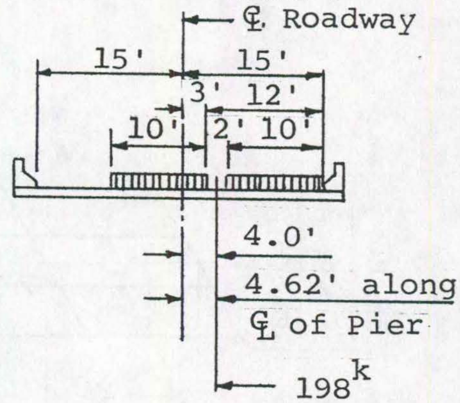
(c) ICE

Pier (70 c.y.)(27)(0.0625) = 118^k

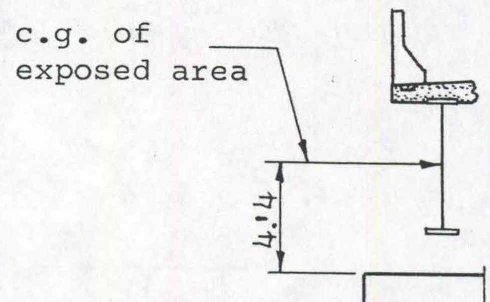
E = 41

159^k

(5) L (No impact load)



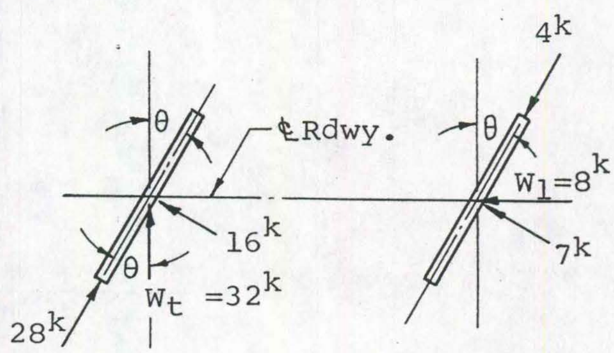
(6) W (Wind on superstructure)



Exposed area = 6.5 □'/,

W_t = 6.5□' x 0.05 k/□' x 99.5 = 32^k

W_l = 6.5 x 0.012 x 99.5 = 8^k

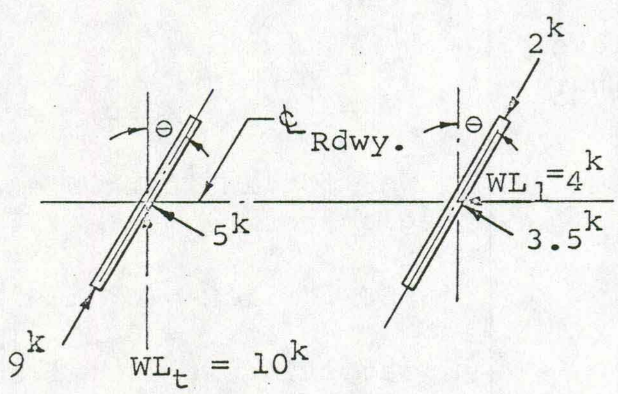


Do not combine the components of W_t and W_l at this stage of calculation. The above arrangement of W_t and W_l is made to produce maximum component Force ⊥ to the centerline of pier. Reverse either the direction of W_t or W_l to determine maximum component force ∥ to the centerline of pier.

(7) WL (Wind on LL)

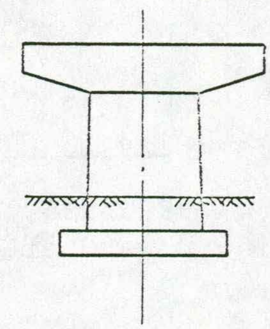
$WL_t = 0.1^{k/'} \times 99'.5 = 10^k$
$WL_l = 0.04 \times 99'.5 = 4^k$

WL_t is applied at 6' above roadway or 44' above bottom of footing. WL_l is applied at bridge seat level.



(8) WOS (Wind on Substructure)

In this design, only the wind acting on the exposed pier area as seen in the front elevation is considered



Exposed area above LW = 530 □'

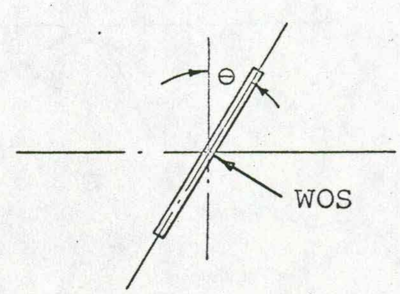
WOS = 530 □' x 0.04 $^{k}/_{□'}$ = 21^k
at 22' from bottom of ftg.

Exposed area above ICE = 384 □'

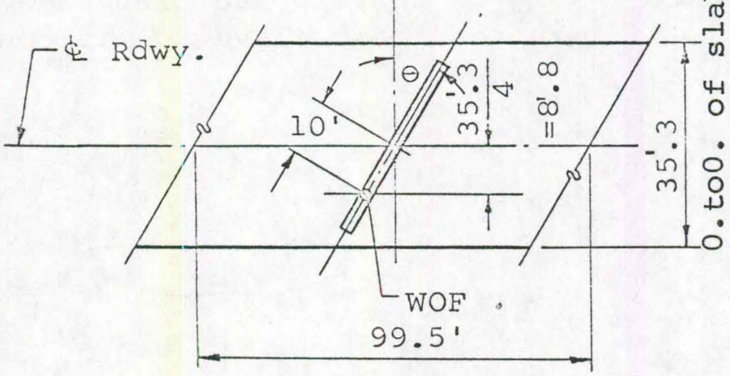
WOS = 384 □' x 0.04 = 15^k
at 26' from bottom of ftg.

Exposed area above HW = 259 □'

WOS = 259 □' x 0.04 = 10^k
at 28' from bottom of ftg.



(9) WOF (Overturning Force)



$WOF = (99.5' \times 35.3)\,0.02\ ^{k}/_{\square'} = 70^{k}$
acting upward at location shown.

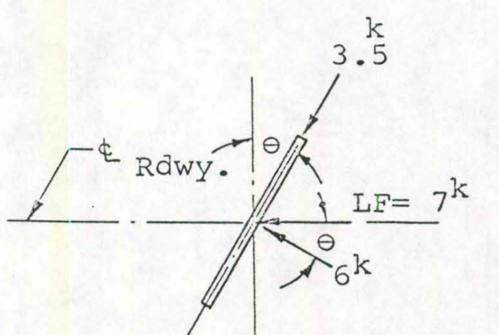
(10) LF

Total LF = [2 (0.64x454' + 18^{k})] 0.05 = 31^{k}

LF to this pier = $\frac{31^{k}}{454'}$ x 99.'5

= 7^{k}

LF is applied at bridge seat level or 32'.5 above bottom of footing.



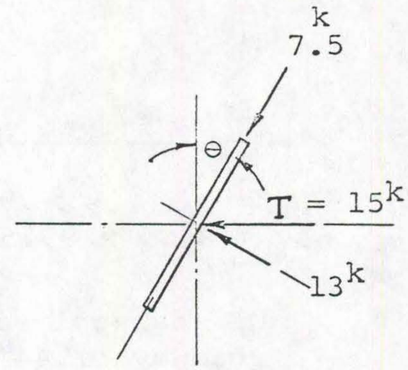
(11) T (Thermal Force)

Super DL

$T = 573^{k} \times 0.25 \times \frac{1''.25}{12''}$

$= 15^{k}$

T is applied at bridge seat level or 32'.5 above bottom of footing.



(12) SF

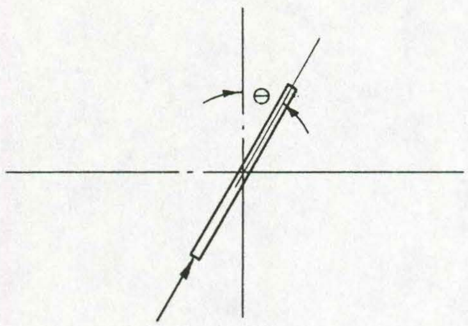
$P = KV^2$

$P = KV^2$
$= \frac{2}{3} \times (4'/\text{Sec})^2 = 11\ ^{\#}/_{\square'}$

SF for LW in this example is neglected.

SF for ICE =(0.011 $^{k}/_{\square'}$)(2.5) (8
= 0.2^{k}
applying at 13' from bottom of ftg.

SF for HW = (0.011 $^{k}/_{\square'}$)(2'.5)(15')
= 0.4^{k}
applying at 16'.5 from bottom of ftg.

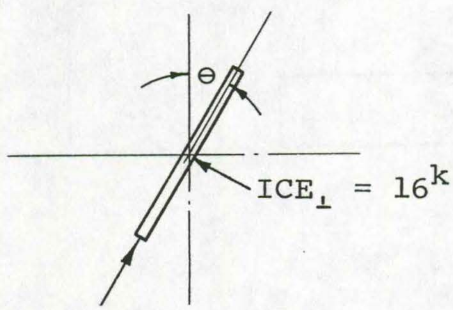


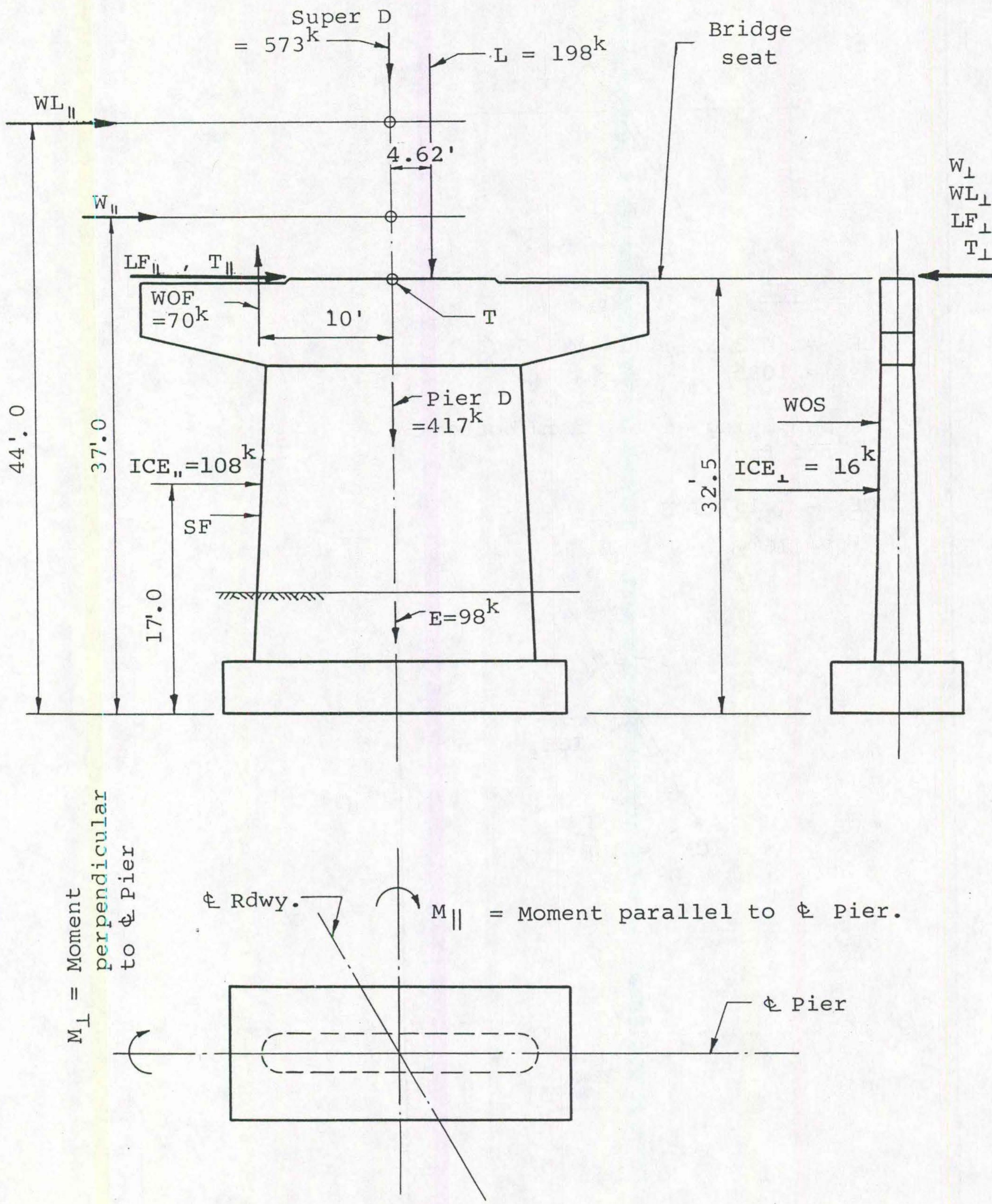
(13) <u>ICE</u>

$ICE_{\parallel} = (0.3\ k/\square'')(144)(2.'5)(1')$
$= 108^k$

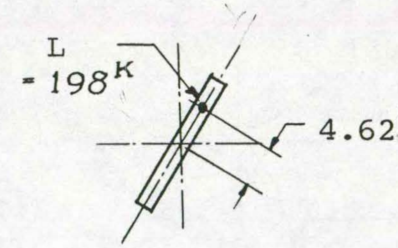
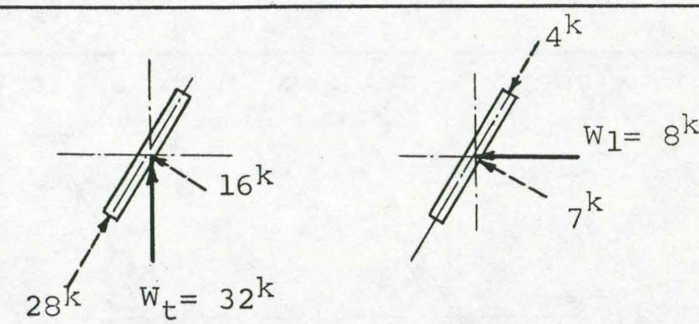
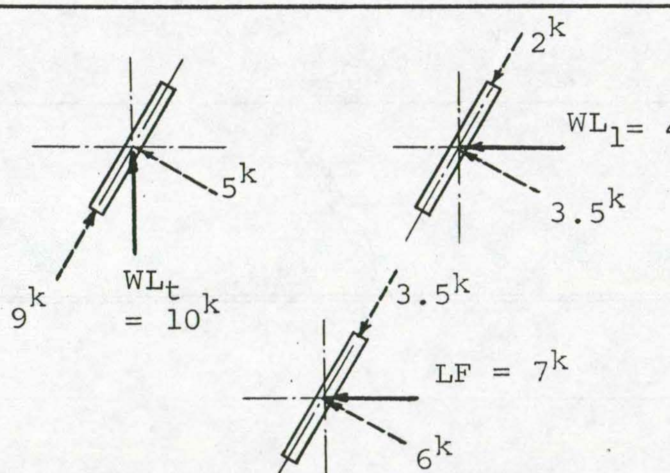
applying at 17' from bottom of ftg.

$ICE_{\perp} = (.15)\ ICE_{\parallel}$
$= 16^k$

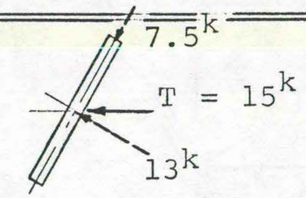
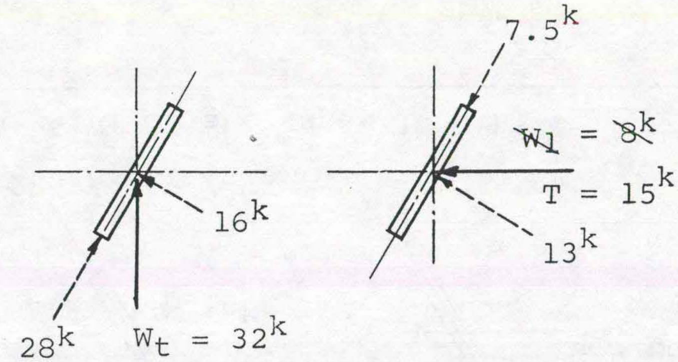
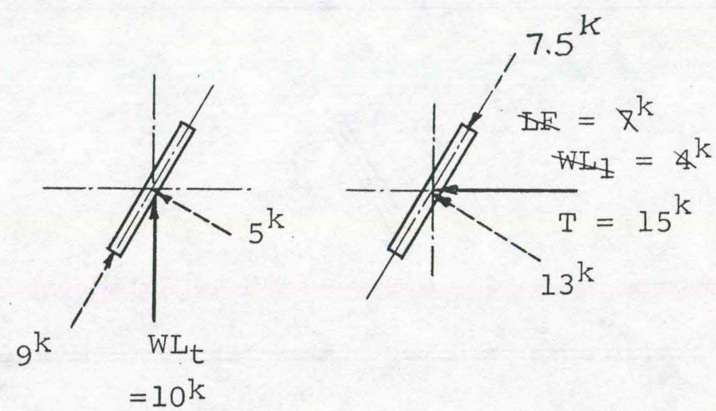



Super D
= 573k
L = 198k
Bridge seat
WL∥
4.62'
W⊥
WL⊥
LF⊥
T⊥
W∥
LF∥ , T∥
WOF
=70k
10'
T
Pier D
=417k
WOS
44'.0
37'.0
ICE∥=108k
32.5
ICE⊥ = 16k
SF
17.0
E=98k
M⊥ = Moment perpendicular to ℄ Pier
℄ Rdwy.
M∥ = Moment parallel to ℄ Pier.
℄ Pier

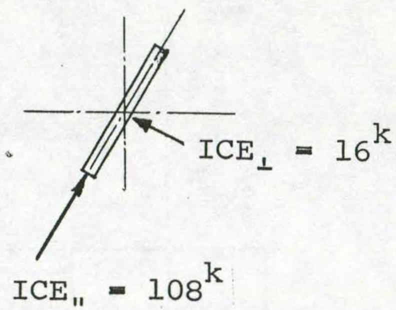
EXP. PIER MAX. M⊥

Gp	Loads	P (Kips)	M ∥ (ft.-Kip)	M⊥ (ft.-kip)	Remark
I	D	573	0	0	
		417	0	0	
	E	98	0	0	
	B (LW)	- 131	0	0	
	L	198	198^kx4.62 = 915	0	
		1,155	915	0	
II	D + E +B (LW)	957	0	0	
	$W_{\parallel}$	0	$(28-4)^k$x37' =888	–	
	$W_{\perp}$	0	–	$(16+7)^k$x32'.5 =748	
	WOS (LW)	0	0	21^kx22' = 462	
	WOF	- 70	70^kx10' = 700	0	
		887	1,588	1,210	W_t & W_l on ave. span length.
III	Gr.I	1,155	915	0	
	$0.3W_{\parallel}$	0	266	–	
	$0.3W_{\perp}$	0	–	224	
	0.3WOS	0	0	139	
	0.3WOF	- 21	210	0	
	$WL_{\parallel}$	0	$(9-2)^k$x44'=308	–	
	$WL_{\perp}$	0	–	$(5+3.5)^k$x32'.5 =276	
	$LF_{\parallel}$	0	-3.5^Kx32'.5=-114	–	
	$LF_{\perp}$	0	–	6^Kx32'.5 = 195	
		1,134	1,585	834	WL_t, WL_l, & LF on ave. span length.

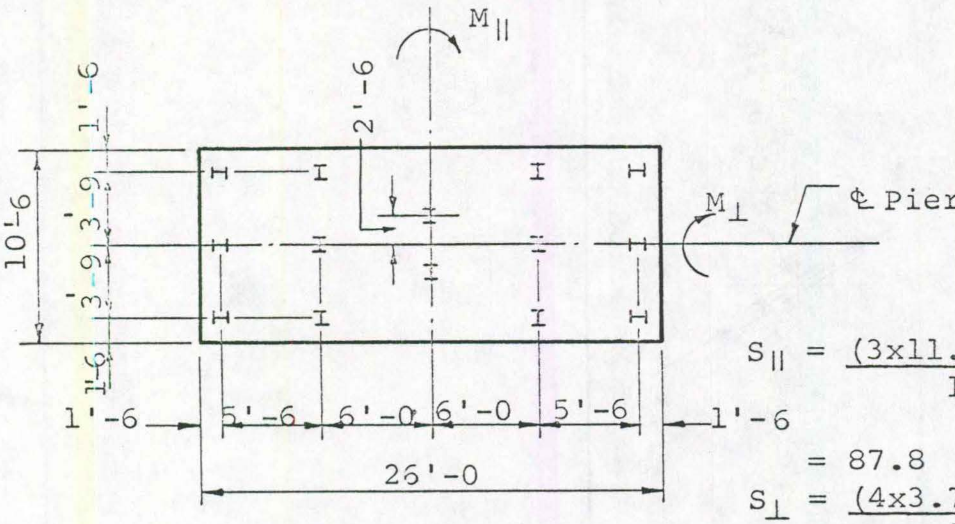
EXP. PIER MAX $M_\perp$

Gp	Loads	P (Kips)	$M_\parallel$ (ft.-Kip)	$M_\perp$ (ft.-Kip)	Remark
IV	Gr I	1,155	915	0	
	$T_\parallel$	0	7.5^Kx 32'.5 =244	-	
	$T_\perp$	0	-	13^Kx32'.5=423	
		1,155	1159	423	
V	D + E + B(LW)	957	0	0	
	$W_\parallel$	0	28^Kx37' = 1,036	-	
	$W_\perp$	0	-	16^Kx32'.5 =520	
	WOS	0	0	462	
	WOF	-70	700	0	
	$T_\parallel$	0	- 244	-	
	$T_\perp$	0	-	13^Kx32'.5= 423	
		887	1,492	1,405	W_l is deleted. See Table VII on P. 18.
VI	D + E + L + B	1,155	915	0	
	0.3 $W_\parallel$	0	311	-	
	0.3 $W_\perp$	0	-	156	
	0.3 WOS	0	0	139	
	0.3 WOF	-21	210	0	
	$WL_\parallel$	0	9^Kx44'=396	-	
	$WL_\perp$	0	-	5^Kx32'.5=162	
	$LF_\parallel$	0	-	-	
	$LF_\perp$	0	-	-	
	$T_\parallel$	0	- 244	-	
	$T_\perp$	0	-	423	
		1,134	1,588	880	LF & WL_l are deleted. See Table VII on P. 18.

EXP. PIER MAX. M⊥

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft.-Kip)	$M_{\perp}$ (ft-kip)	Remark
VIII	D + E	1,088	0	0	
	B (ICE)	- 159	0	0	
	LL	198	915	0	
	ICE	0	$108^k \times 17' = 1,836$	$16^k + 17 = 272$	
	SF	0	$0.2^k \times 13' = 3$	0	
		1,127	2,754	272	
IX	D + E	1,088	0	0	
	B (ICE)	- 159	0	0	
	$W_{\parallel}$	0	888	-	
	$W_{\perp}$	0	-	748	
	WOS (ICE)	0	0	$15^k \times 26' = 390$	
	WOF	- 70	700	0	
	ICE	0	1,836	272	
	SF	0	3	0	
		859	3,427	1,410	

14-HP10x42 Piling



$$S_{\parallel} = \frac{(3\times11.5^2 + 3\times6^2)(2)}{11.5}$$

$$= 87.8$$

$$S_{\perp} = \frac{(4\times3.75^2 + 2.5^2)(2)}{3.75}$$

$$= 33.3$$

EXP. PIER MAX M⊥

Gp	$\frac{P}{14}$	$\frac{M_{\parallel}}{87.8}$	$\frac{M_{\perp}}{33.3}$	Total Max.	Total Min.	100% Max.	100% Min.	
I	83[k]	10[k]	0[k]	93[k]	73[k]	93[k]	73[k]	1
II	63	18	36	117	9	94	7	1.25
III	81	18	25	124	38	99	30	1.25
IV	83	13	13	109	57	87	46	1.25
V	63	17	42	122	4	87	3	1.4
VI	81	18	26	125	37	89	26	1.4
VIII	81	31	8	120	42	86	30	1.4
IX	61	39	42	142	-20*	95	-13*	1.5

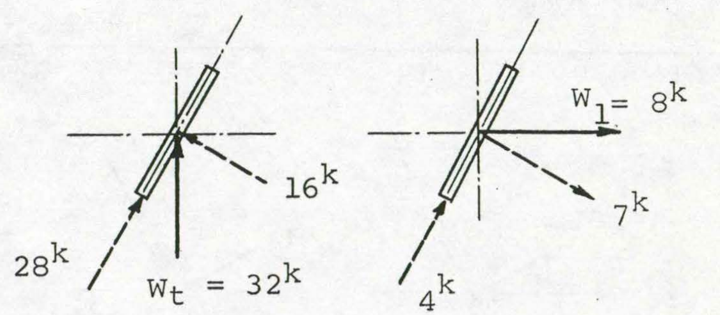
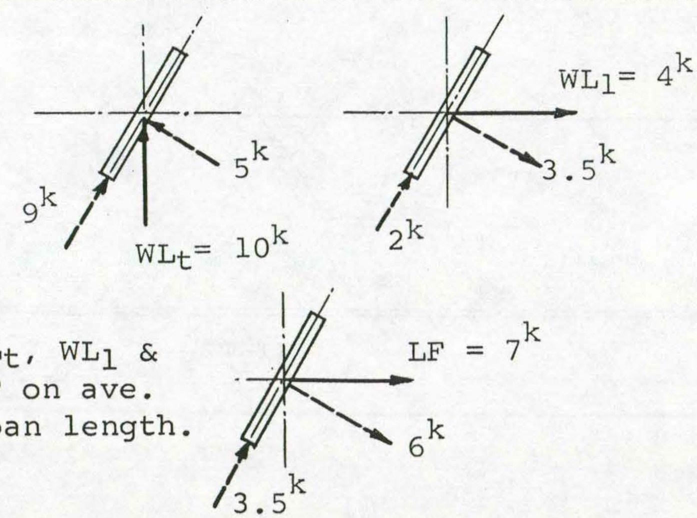
Design allowable bearing for HP10x42 = 110^k

Design allowable uplift $= \frac{(6\times10")^{\Delta}(12")(15\ psi)}{1{,}000} = 10.8^k$ for 12" piling embedment in ftg.

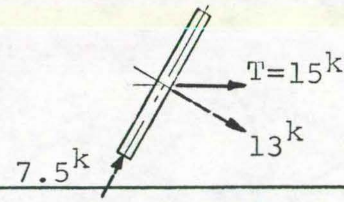
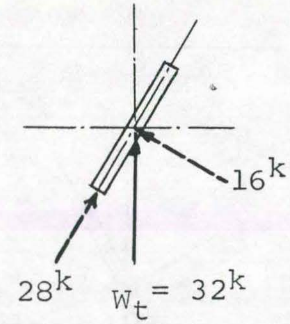
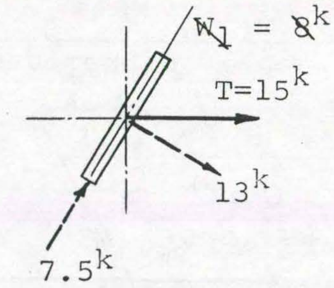
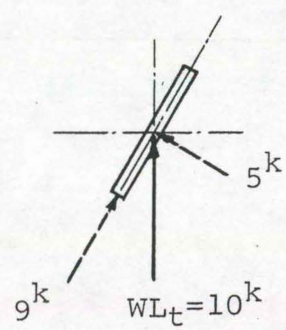
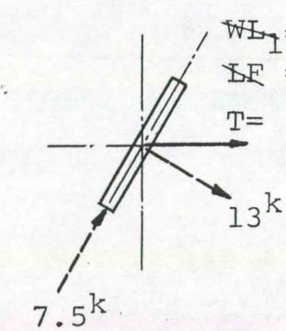
* Uplift

^Δ Approx. pile surface area per inch of embedment.

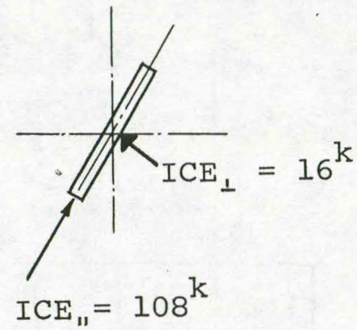
EXP. PIER MAX $M_{\parallel}$

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft.-Kip)	$M_{\perp}$ (ft.-kip)	Remark
I	D	573	0	0	
		417	0	0	
	E	98	0	0	
	B (LW)	- 131	0	0	
	LL	198	198^kx4.62 = 915	0	
		1,155	915	0	
II	D + E				
	+B (LW)	957	0		
	$W_{\parallel}$	0	$(28+4)^K$x37'=1,184	–	
	$W_{\perp}$	0	–	$(16-7)^K$x32'.5 =293	
	WOS (LW)	0	0	21^kx22' = 462	
	WOF	- 70	70^kx10' = 700	0	
		887	1,884	755	W_t & W_l on ave. span length
III	Gr.I	1,155	915	0	
	0.3$W_{\parallel}$	0	355	–	
	0.3$W_{\perp}$	0	–	88	
	0.3WOS	0	0	139	
	0.3WOF	- 21	210	0	
	$WL_{\parallel}$	0	$(9+2)^K$x44'=484	–	
	$WL_{\perp}$	0	–	$(5-3.5)^K$x32'.5=49	
	$LF_{\parallel}$	0	3.5^Kx32'.5=114	–	
	$LF_{\perp}$	0	–	-6^Kx32'.5 =-195	
		1,134	2,078	81	WL_t, WL_l & LF on ave. span length.

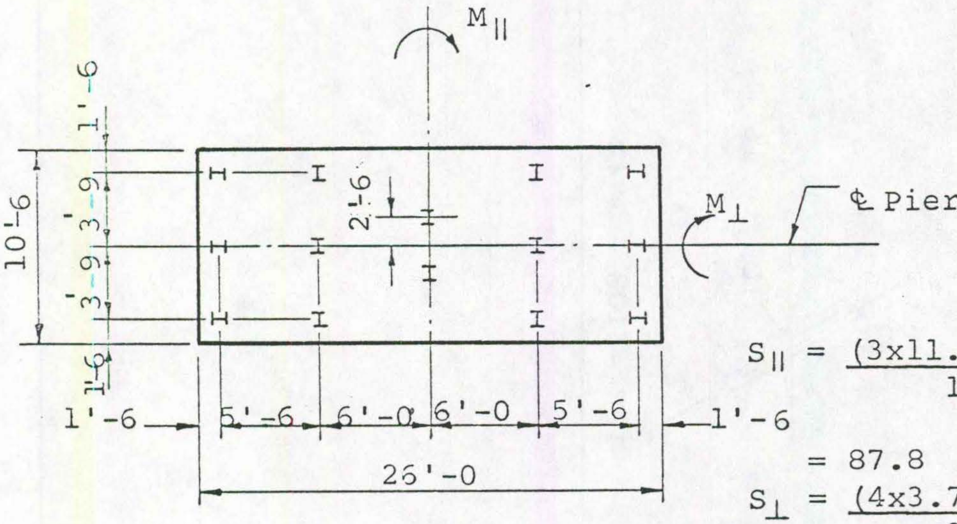
EXP. PIER MAX. $M_{\parallel}$

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft.-Kip)	$M_{\perp}$ (ft.-Kip)	Remark
IV	Gr I	1,155	915	0	
	$T_{\parallel}$	0	7.5^Kx32'.5=244	-	
	$T_{\perp}$	0	-	13^Kx32'.5 = 423	
		1,155	1,159	423	
V	D + E + B(LW)	957	0	0	
	$W_{\parallel}$	0	28^Kx37' = 1,036	-	 
	$W_{\perp}$	0	-	16^Kx32'.5= 520	
	WOS	0	0	462	
	WOF	-70	700	0	
	$T_{\parallel}$	0	7.5^Kx32'.5=244		
	$T_{\perp}$	0	-	-13^Kx32'.5=-423	
		887	1,980	559	W_l is deleted. See Table VII on P. 18.
VI	D + E + L + B	1,155	915	0	
	0.3 $W_{\parallel}$	0	311	-	 
	0.3 $W_{\perp}$	0	-	156	
	0.3 WOS	0	0	139	
	0.3 WOF	-21	210	0	
	$WL_{\parallel}$	0	9^Kx44' =396	-	
	$WL_{\perp}$	0	-	5^Kx32'.5= 162	
	LF $\parallel$	0	-	-	
	LF $\perp$	0	-	-	
	$T_{\parallel}$	0	244	-	
	$T_{\perp}$	0	-	-423	
		1,134	2,076	34	LF & WL_l are deleted. See Table VII on P. 18

EXP PIER MAX. $M_{\parallel}$

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft.-Kip)	$M_{\perp}$ (ft-kip)	Remark
VIII	D + E	1,088	0	0	
	B (ICE)	- 159	0	0	
	L	198	915	0	
	ICE	0	108^kx17' = 1,836	16x17=272	
	SF	0	0.2^k x13 = 3	0	
		1,127	2,754	272	
IX	D + E	1,088	0	0	
	B(ICE)	- 159	0	0	
	$W_{\parallel}$	0	1,184	–	
	$W_{\perp}$	0	–	293	
	WOS (ICE)	0	0	15^kx26'=390	
	WOF	- 70	700	0	
	ICE	0	1,836	272	
	SF	0	3	0	
		859	3,723	955	

14- HP10x42 Piling



$$S_{\parallel} = \frac{(3 \times 11.5^2 + 3 \times 6^2)(2)}{11.5}$$

$$= 87.8$$

$$S_{\perp} = \frac{(4 \times 3.75^2 + 2.5^2)(2)}{3.75}$$

$$= 33.3$$

EXP. PIER MAX. M||

Gp	P/14	M||/87.8	M⊥/33.3	Total Max.	Total Min.	100% Max.	100% Min.	
I	83k	10k	0k	93k	73k	93k	73k	1
II	63	21	23	107	19	86	15	1.25
III	81	24	2	107	55	86	44	1.25
IV	83	13	13	109	57	87	46	1.25
V	63	23	17	103	23	74	16	1.4
VI	81	24	1	106	56	76	40	1.4
VIII	81	31	8	120	42	86	30	1.4
IX	61	42	29	132	-10*	88	-7*	1.5

Design allowable bearing for HP10x42 = 110^k

Design allowable uplift = $\frac{(6 \times 10")^{\Delta}(12")(15\text{ psi})}{1,000} = 10.8^k$ for 12" piling embedment in ftg.

* Uplift

△ Approx. pile surface area per inch of embedment.

Pier # 3 - Fixed Pier

Loadings

(1) Super D 573^k

(2) Pier D
(106 c.y.)(27)(0.15) = 429^k

(3) E
(39 c.y.)(27)(0.1) = 105^k

(4) Bouyancy

(a) LW

Pier (57 c.y.)(27)(0.0625)= 96^k
E (39 c.y.)(27)(2/3)(0.0625)= 44

140^k

(b) HW

Pier (91 c.y.)(27)(0.0625)=154^k
E = 44

198^k

(c) ICE

Pier (73 c.y.)(27)(0.0625)=123^k
E = 44

167^k

(5) L (No impact load)

See Sh. 2E

(6) W_l, (7) WL_l & (10) LF_l shall be determined on the basis of (1) overall bridge length and (2) average span length of two spans adjacent to Pier #3.

(6) W

W_t (avg. span length)
$= 6.5\ \square' \times 0.05^K/\square' \times 99'.5 = 32^K$

W_l (bridge length)

$= 6.5\square' \times 0.012^k/\square' \times 454'$

$= 35.4^k$

W_l (average span length)

$= 6.5\square' \times 0.012^k/\square' \times 99'.5$

$= 7.8^k$

(7) WL

WL_t (avg. span length)
$= 0.1^K/' \times 99'.5 = 10^K$

WL_l (bridge length)

$= 0.04\ ^k/' \times 454' = 18^k$

WL_1 (average span length)

$= 0.04\ ^{k}/' \times 99'.5 = 4^{k}.0$

(8) WOS and (9) WOF are the same as for Pier #2.

(10) LF

LF (Bridge length)

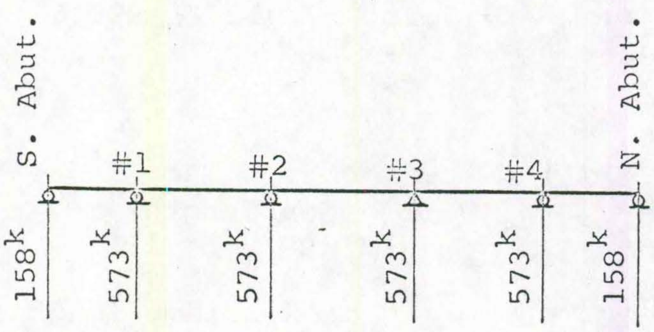
$= [2(0^{k}.64 \times 454' + 18^{k})] \times 0.05$

$= 31^{k}$

LF (average span length)

$= 31^{k} \times \frac{99'.5}{454'} = 7^{k}$

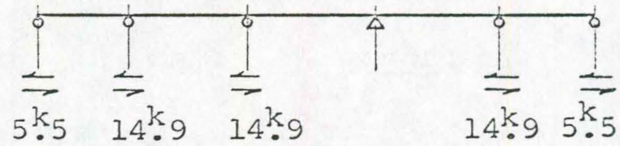
(11) T



DL Reactions

T at each S and N Abut. =
$158^{k} \times 25\% \times \frac{1.25}{9} = 5^{k}.5$

T at each Pier #1, 2 & 4 =
$573^{k} \times 25\% \times \frac{1.25}{12} = 14^{k}.9$

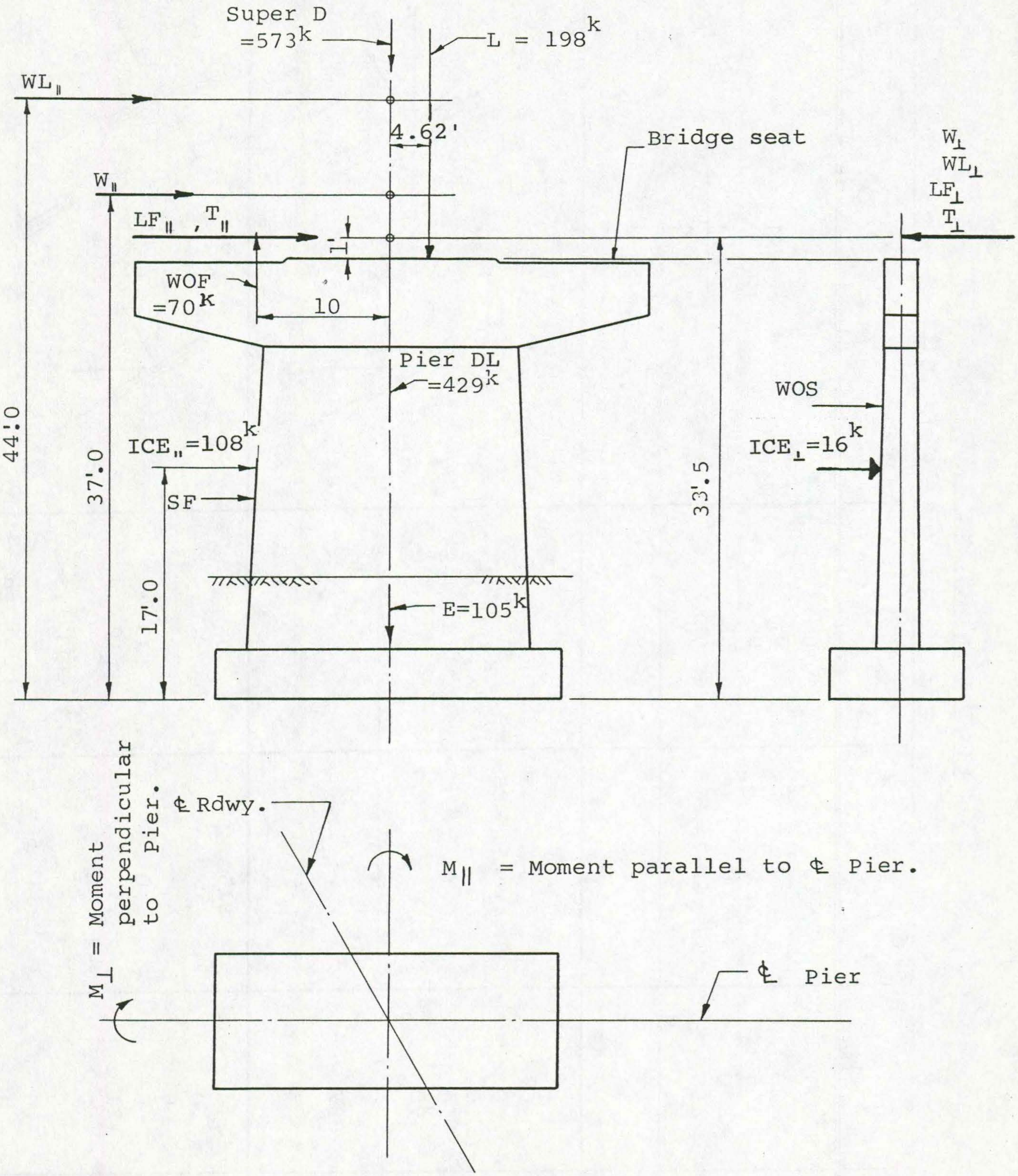


T

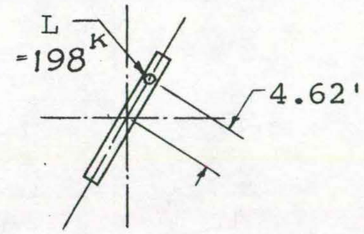
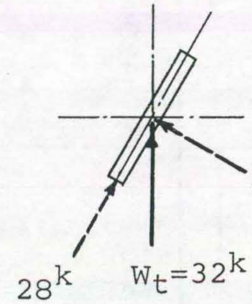
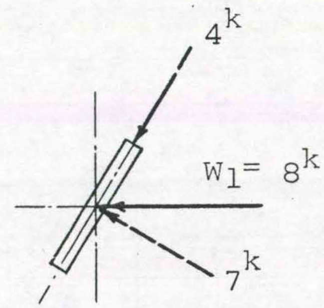
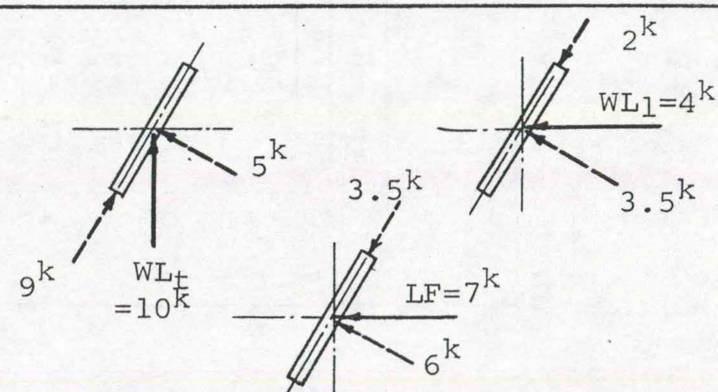
Unbalanced thermal=$14^{k}.9$

T applies at 1' above bridge seat level or 33'.5 above bottom of footing.

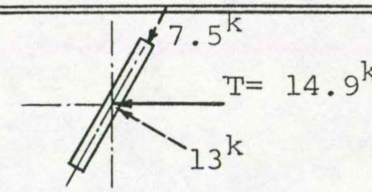
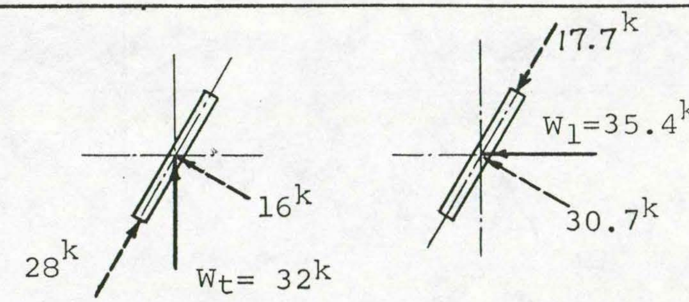
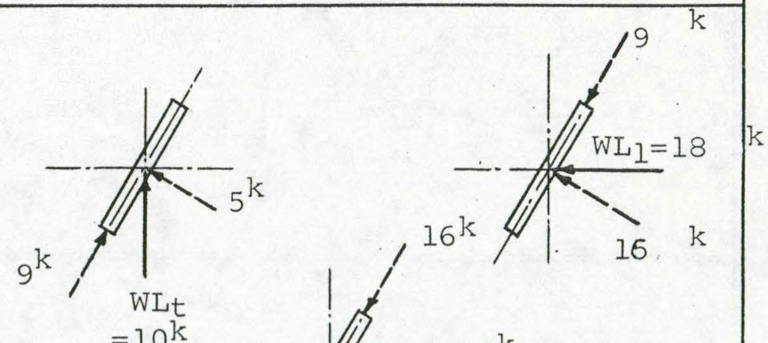
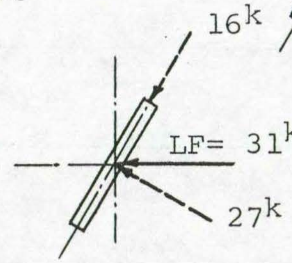
(12) SF and (13) ICE are the same as for Pier #2.


Super D
=573k
L = 198k
WL∥
4.62'
Bridge seat
W⊥
WL⊥
LF⊥
T⊥
W∥
LF∥, T∥
1
WOF
=70k
10
Pier DL
=429k
WOS
ICE∥=108k
ICE⊥=16k
SF
44'0
37'.0
33'.5
17'.0
E=105k
℄ Rdwy.
M⊥ = Moment perpendicular to Pier.
M∥ = Moment parallel to ℄ Pier.
℄ Pier

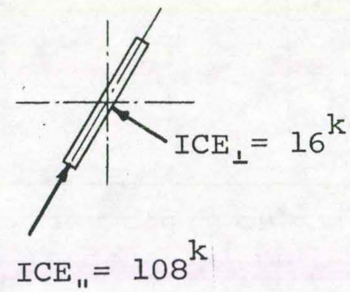
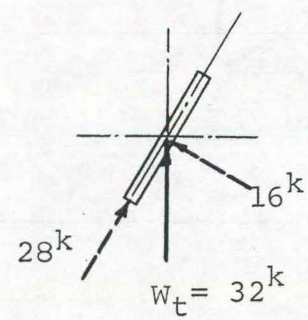
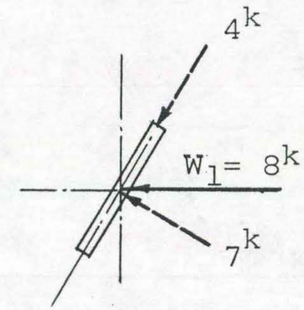
FIXED PIER MAX. $M_\perp$

Gp	Loads	P (Kips)	$M_\parallel$ (ft-Kip)	$M_\perp$ (ft-kip)	Remark
I	D	573	0	0	
		429	0	0	
	E	105	0	0	
	B (LW)	- 140	0	0	
	L	198	198^{k}x4.62 = 915	0	
		1,165	915	0	
II	D + E +B (LW)	967	0	-	W_t & W_l on ave. span length.  
	$W_\parallel$	0	$(28-4)^{K}$x37'=888	—	
	$W_\perp$	0	—	$(16+7)^{K}$x33'.5=771	
	WOS (LW)	0	0	21^{k}x22' = 462	
	WOF	-70	70^{k}x10' = 700	0	
		897	1,588	1,233	
III	Gr. I	1,165	915	0	
	$0.3W_\parallel$	0	266	—	
	$0.3W_\perp$	0	—	231	
	0.3WOS	0	0	139	
	0.3WOF	-21	210	0	
	$WL_\parallel$	0	$(9-2)^{K}$x44' = 308	—	
	$WL_\perp$	0	—	$(5+3.5)^{K}$x33'.5=285	
	$LF_\parallel$	0	-3.5^{K}x33'.5 = -117	—	
	$LF_\perp$	0	—	6^{K}x33'.5=201	
		1,144	1,582	856	WL_t, WL_l & LF on ave. span length.

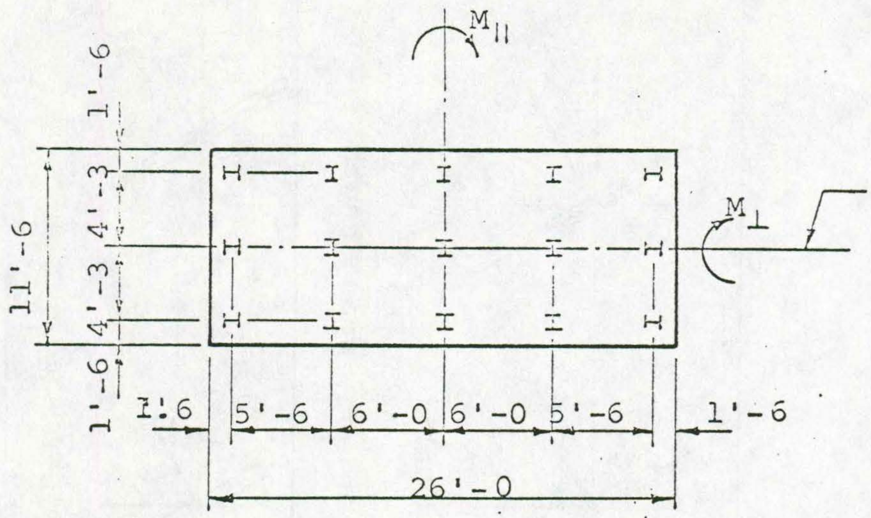
FIXED PIER MAX $M_\perp$

Gp	Loads	P (Kips)	$M_\parallel$ (ft-Kips)	$M_\perp$ (ft-Kips)	Remark
IV	Gr I	1,165	915	0	
	$T_\parallel$	0	7.5^kx33.5= 251	—	
	$T_\perp$	0	—	13^Kx33'.5=436	
		1,165	1,166	436	
V	D +E+ B(LW)	967	0	0	
	$W_\parallel$	0	$(28-17.7)^K$37'=381	—	
	$W_\perp$	0	—	$(16+30.7)^K$33'.5=1,565	
	WOS	0	0	462	
	WOF	-70	700	0	
	$T_\parallel$	0	-7.5^kx33.5=-251	—	
	$T_\perp$	0	—	13^Kx33'.5=436	
		897	830	2,463	W_t on avg. span length. W_l on bridge length. See Note ② on P. 14.
VI	D +E+ L +B	1,165	915	0	
	0.3$W_\parallel$	0	0.3x381= 114	—	
	0.3$W_\perp$		—	0.3x1,565 =470	
	0.3WOS	0	0	139	
	0.3WOF	-21	210	0	
	$WL_\parallel$	0	0	—	
	$WL_\perp$	0	0	$(5+16)^K$33'.5=704	
	$LF_\parallel$	0	-16^kx33'.5=-536	—	
	$LF_\perp$	0	—	27^Kx33'.5 = 905	
	$T_\parallel$	0	-7.5^Kx33'.5=-251	—	
	$T_\perp$	0	—	13^Kx33'.5 = 436	
		1,144	452	2,654	WL_t on avg. span length. WL_l & LF on bridge length. See Note ② on P. 14

FIXED PIER MAX. M⊥

Gp	Loads	P (Kips)	M∥ (ft.-kips)	M⊥ (Ft.-Kips)	Remark
VIII	D + E	1,107	0	0	
	B(ICE)	- 167	0	0	
	L	198	915	0	
	ICE	0	108k x17'=1,836	16x17=272	
	SF	0	0.2k x13k = 3	0	
		1,138	2,754	272	
IX	D + E	1,107	0	0	
	B (ICE)	- 167	0	0	
	W∥	0	888	—	
	W⊥	0	—	771	
	WOS (ICE)	0	0	15k x26' = 390	
	WOF	-70	700	0	
	ICE	0	1,836	272	
	SF	0	3	0	
		870	3,427	1,433	

15 - HP10x42 Pilings



$S_{\parallel} = 87.8$

$S_{\perp} = \frac{(5 \times 4.25^2)(2)}{4.25}$

$= 42.5$

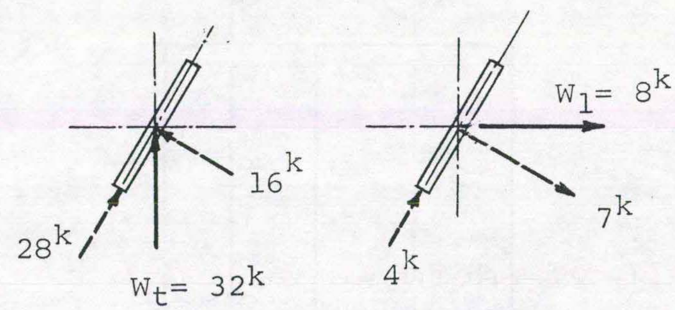
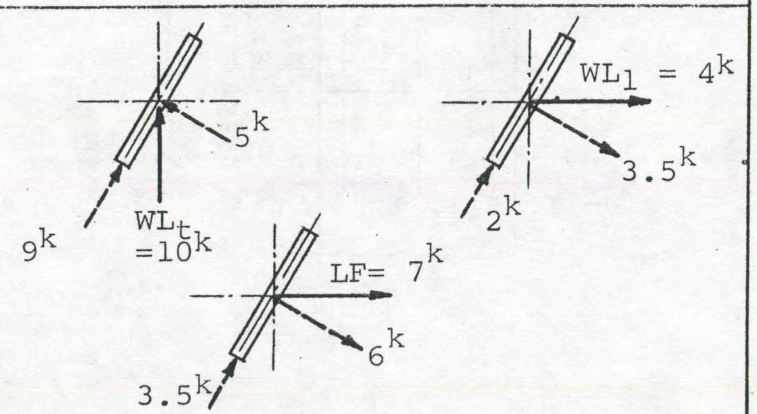
FIXED PIER MAX. $M_{\perp}$

G_p	$\frac{P}{15}$	$\frac{M_{\parallel}}{87.8}$	$\frac{M_{\perp}}{42.5}$	Total Max.	Total Min.	100% Max.	100% Min.	
I	78^k	10^k	0^k	88^k	68^k	88^k	68^k	1
II	60	18	29	107	13	86	10	1.25
III	76	18	20	114	38	91	30	1.25
IV	78	13	10	101	55	81	44	1.25
V	60	9	58	127	-7*	91	-5*	1.4
VI	76	5	62	143	9	102	6	1.4
VIII	76	31	6	113	39	81	28	1.4
IX	58	39	34	131	-15*	87	-10*	1.5

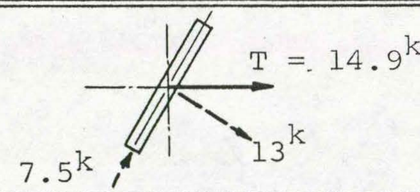
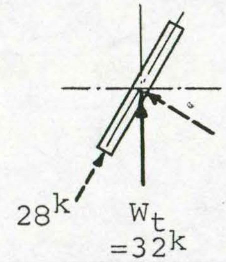
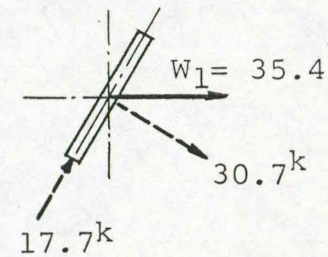
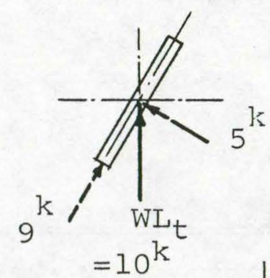
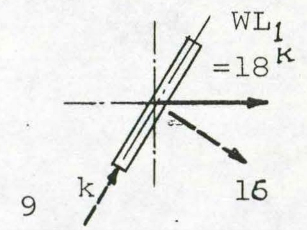
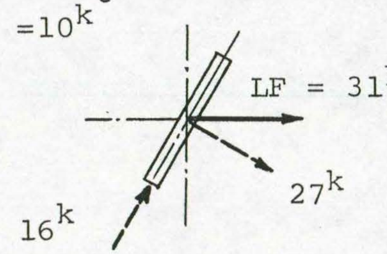
Design allowable bearing for HP10x42 = 110^k
Design allowable uplift = 10.8^k for 12" piling embedment in ftg.

* Uplift

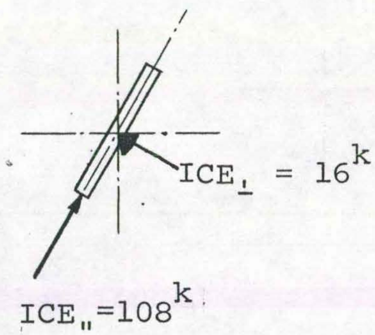
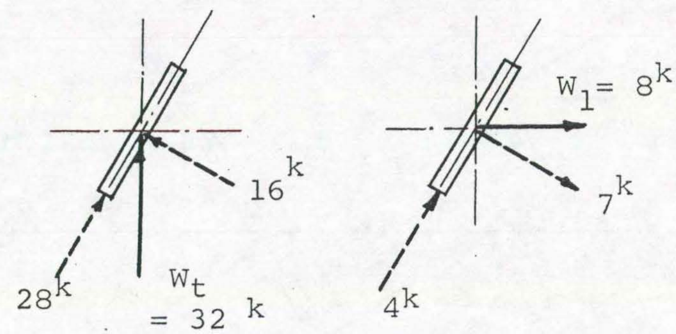
FIXED PIER MAX. $M_{\parallel}$

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft-Kip)	$M_{\perp}$ (ft-kip)	Remark
I	D	573	0	0	
		429	0	0	
	E	105	0	0	
	B (LW)	- 140	0	0	
	L	198	198^{k}x4.62=915	0	
		1,165	915	0	
II	D + E				
	+B (LW)	967	0	-	
	$W_{\parallel}$	0	$(28+4)^{K}$ 37'=1,185	—	
	$W_{\perp}$	0	—	$(16-7)^{K}$33'.5 =302	
	WOS (LW)	0	0	21^{k}x22' = 462	
	WOF	-70	70^{k}x10' = 700	0	
		897	1,885	764	W_t & W_l on avg. span length
III	Gr.I	1,165	915	0	
	$0.3W_{\parallel}$	0	356	—	
	$0.3W_{\perp}$	0	—	91	
	0.3WOS	0	0	139	
	0.3WOF	-21	210	0	
	$WL_{\parallel}$	0	$(9+2)^{K}$x44' = 484	—	
	$WL_{\perp}$	0	—	$(5-3.5)^{K}$x33'.5=50	
	LF $\parallel$	0	3.5^{K}x33.'5 = 117	—	
	LF $\perp$	0	—	-6x33'.5 = -201	
		1,144	2,082	79	WLt, WL_l & LF on avg. span length

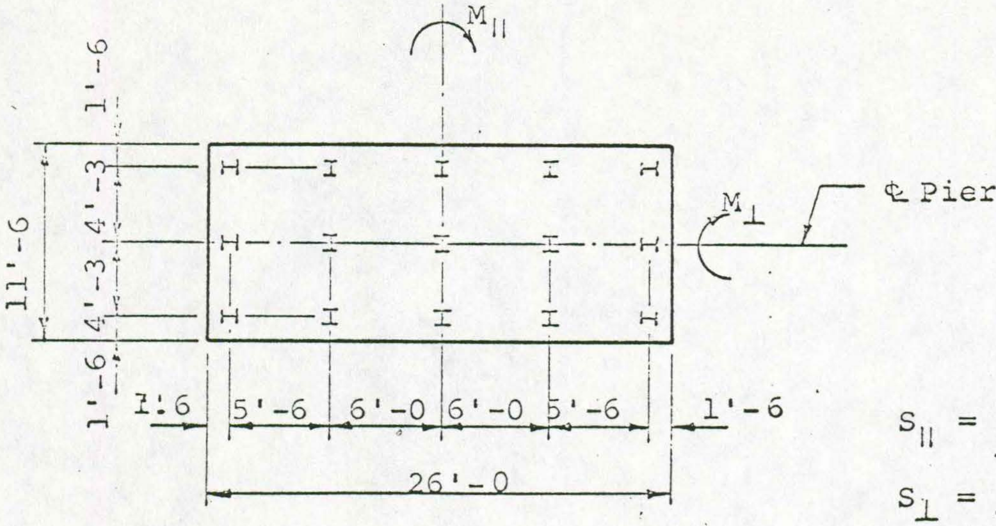
FIXED PIER MAX. $M_{\parallel}$

Gp	Loads	P (Kips)	$M_{\parallel}$ (ft-Kips)	$M_{\perp}$ (ft-Kips)	Remark
IV	Gr I $T_{\parallel}$ $T_{\perp}$	1,165 0 0 1,165	915 $7.5^{k} x 33.5 =$ 251 — 1,166	0 — $13^{K} x 33'.5 =$ 436 436	
V	D +E+ B(LW) $W_{\parallel}$ $W_{\perp}$ WOS WOF $T_{\parallel}$ $T_{\perp}$	 967 0 0 0 -70 0 0 897	 0 $(28+17.7)^{K}$ 37'= 1,691 0 700 $7.5^{k} x 33.5 =$ 251 — 2,642	 0 — $(30.7-16)^{K}$ 33'.5=492 462 0 — 13^{K} 33'.5 = 436 1,390	W_t on avg. span length W_l on bridge length.
VI	D +E+ L +B $0.3W_{\parallel}$ $0.3W_{\perp}$ 0.3WOS 0.3WOF $WL_{\parallel}$ $WL_{\perp}$ $LF_{\parallel}$ $LF_{\perp}$ $T_{\parallel}$ $T_{\perp}$	1,165 0 0 0 -21 0 0 0 0 0 0 1,144	915 508 — 0 210 $(9+9)^{K}$44'= 792 — 16^{k} x 33'.5 =536 — 251 — 3,212	0 — 148 139 0 — $(16-5)^{K}$33'.5=369 — 27^{K}x33'.5 = 905 — 436 1,997	WL_t on avg. span length. WL_l & LF on bridge length. See Note ② on P. 14

FIXED PIER MAX $M_\perp$

Gp	Loads	P (Kips)	$M_\parallel$ (ft.-kips)	$M_\perp$ (Ft.-Kips)	Remark
VIII	D + E	1,107	0	0	
	B(ICE)	- 167	0	0	
	L	198	915	0	
	ICE	0	$108^k \times 17' = 1,836$	$16 \times 17 = 272$	
	SF	0	$0.2^k \times 13^k = 3$	0	
		1,138	2,754	272	
IX	D + E	1,107	0	0	
	B (ICE)	- 167	0	0	
	$W_\parallel$	0	1,185	—	
	$W_\perp$	0	—	302	
	WOS (ICE)	0	0	$15^k \times 26' = 390$	
	WOF	-70	700	0	
	ICE	0	1,836	272	
	SF	0	3	0	
		870	3,724	964	

15 - HP10x42 Pilings



$S_{\parallel} = 87.8$

$$S_{\perp} = \frac{(5 \times 4.25^2)(2)}{4.25}$$

$= 42.5$

FIXED PIER MAX $M_{\parallel}$

Gp	$\frac{P}{15}$	$\frac{M_{\parallel}}{87.8}$	$\frac{M_{\perp}}{42.5}$	Total		100%		
				Max.	Min.	Max.	Min.	
I	78^k	10^k	0^k	88^k	68^k	88^k	68^k	1
II	60	22	18	100	20	80	16	1.25
III	76	24	2	102	50	82	40	1.25
IV	78	13	10	101	55	81	44	1.25
V	60	30	33	123	-3 *	88	- 2 *	1.4
VI	76	37	47	160	- 8 *	114	- 6*	1.4
VIII	76	31	6	113	39	81	28	1.4
IX	58	42	23	123	- 7 *	82	- 5*	1.5

Design allowable bearing for HP10x42 = 110^k
Design allowable uplift = 10.8^k for 12" piling embedment in ftg.

* Uplift