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Design Submittal

GUADALUPE ROAD BRIDGE OVER RWCD FLOODWAY
Maricopa County Flood Control District
Contract FCD 83-28

CANNON & ASSOCIATES, INC.
CONSULTING ENGINEERS

October 8, 1984

Cannon & Associates, Inc.

Consulting Engineers
406 South 4th Avenue
Tucson, Arizona 85701
(602) 792-2200

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Cannon & Associates, Inc.
Consulting Engineers
Tucson, Arizona

October 8, 1984

Cost Estimate

GUADALUPE ROAD BRIDGE OVER RWCD FLOODWAY

Item No.	Description	Unit	Quantity	Unit Cost	Cost
415	Bridge Guard Rail	L.F.	60	$15	$ 900
502-1	24"Ø Drilled Shaft	L.F.	278	$40	11,120
502-2	30"Ø Drilled Shaft	L.F.	280	$50	14,000
505-1	Concrete f'c=4500 psi	C.Y.	413	$200	82,600
505-2	Concrete f'c=4000 psi	C.Y.	184	$180	33,120
505-3	Concrete f'c=3000 psi	C.Y.	80	$150	12,000
505-5	Reinforcing Steel	Lbs.	142,875	$0.35	50,006
	TOTAL COST				$203,746

Cost/SF = $31.30/SF

(SF = 55.17' x 118')

Cannon & Associates, Inc.
Consulting Engineers
Tucson, Arizona

October 9, 1984

Draft Copy

CONSTRUCTION SPECIAL PROVISIONS
FOR
GUADALUPE ROAD BRIDGE
AT RWCD FLOODWAY
CONTRACT FCD 83-28

LOCATION OF THE WORK: This project is located on Guadalupe Road at the RWCD Floodway, 150 feet west of Power Road.

PROPOSED WORK: The work consists of constructing a concrete bridge, approaches and other miscellaneous items of work required for the completion of the project.

SPECIFICATIONS: The work embraced herein and as shown on the plans for the construction of this project shall be done in accordance with the Maricopa Association of Governments Uniform Standard Specifications for Public Works Construction dated 1979 and the current revisions thereto together with the Maricopa County Highway Department Supplement to the Uniform Standard Specifications adopted August 3, 1981 and October 19, 1981 and the Construction Special Provisions contained herein.

PROGRESS SCHEDULE: The Contractor shall submit his proposed work progress schedule to the Engineer for approval before starting the work.

ITEM COMMENTS: The herein contained Construction Special Provisions supplement the Uniform Standard Specifications and the Maricopa County Highway Department Supplement to the Uniform Standard Specifications; however, in case of conflict these Special Provisions supersede the Uniform Standard Specifications and the Maricopa County Supplement.

MATERIAL: Select material, aggregate base and mineral aggregate shall be obtained from commercial sources. The Contractor shall pay all royalties or any other charges or expenses incurred in connection with the securing and hauling of the material. The Contractor will be required to furnish the Engineer with a list of his proposed commercial sources prior to use, and shall present satisfactory evidence that the material produced from any commercial source will meet the specifications as noted and these Special Provisions for such material.

SOIL BORINGS: Soil boring information is available for review at the Flood Control District of Maricopa County.

WORK STANDARDS: The Contractor shall comply with Sections 103 and 107 of the Contract Work Hours and Safety Standards Act (40 U.S.C. 327-330) as supplemented by Department of Labor Regulations (29 CFR Part 5).

WATER, LIGHT, POWER, HEAT, TELEPHONE: All water for construction purposes, drinking water, lighting, temporary electric power, heat and telephone service shall be arranged for and provided for the requirements of the work by the Contractor at his expense.

SUBSECTION 101.2 - DEFINITIONS AND TERMS: Change the definition of Budget Project to read as follows: A project financed by funds set aside in the annual budget or otherwise approved by the Board of Directors of the Flood Control District of Maricopa County.

Change the definition of Engineer to read as follows: The Chief Engineer and General Manager of the Flood Control District of Maricopa County acting directly or through his duly authorized representative.

Change the definition of Owner to read as follows: The Flood Control District of Maricopa County, acting through its legally constituted officials, officers or employees.

SECTION 102 - ADDENDUMS & SUBMISSION OF BIDDING SCHEDULE: It shall be the responsibility of prospective bidders to determine, prior to submission of a bid, if any addendums have been issued. This may be accomplished by calling 602-262-1501. Any addendum issued, if not already bound into the Special Provisions, must be included as a part of the Special Provisions and any quantities on the Bidding Schedule requiring change shall be adjusted by pen and ink, to the new figure.

Bids which do not include appropriate addendums and show appropriate changes to the Bidding Schedule shall be invalid.

SECTION 102.5 - PREPARATION OF PROPOSAL: The bidder's Arizona State Contractor's License number and classification need not be shown on the proposal. The possession of such a license is no longer a bidding requirement; however, the Contractor may be required to provide certification of prior satisfactory completion for similar construction.

SECTION 103.6 - CONTRACTOR'S INSURANCE: Concurrently with the execution of the contract, the Contractor shall furnish a Certificate of Insurance. The types of insurance and the limits of liability shall be as indicated thereon, i.e.,

$1,000,000 bodily injury per person
$5,000,000 bodily injury each occurrence
$1,000,000 property damage

SUBSECTION 104.2.4 - ALTERATION OF WORK - BRIDGE DESIGN: Any request for change in the design of any bridge member; i.e., beam thickness on the canal bridge, requested by the Contractor must be accomplished by calculations indicating that the requested change conforms to the design standards as indicated by the plans.

SECTION 105.2 - PLANS AND SHOP DRAWINGS: Shop drawings shall be submitted as follows:

- Initial submittal for review and/or approval - Three (3) sets of plans of which one (1) will be returned to the Contractor.

Final submittal for approval - Five (5) sets of plans of which two (2) sets will be returned to the Contractor.

SECTION 105.6 - COOPERATION WITH UTILITIES: When the Contractor's operations result in damage to any utility, the location of which has been brought to his attention, he shall assume full responsibility for such damage.

The following telephone numbers as indicated should put the Contractor in contact with the proper personnel:

Mountain Bell Telephone Company.....263-3219
Salt River Project.....273-2201
Arizona Public Service.....271-7014
Location Staking (A.P.S., Mtn. Bell, S.R.P.).....Blue Stake..263-1100
Roosevelt Water Conservation District.....963-3414
City of Mesa.....834-2516
Town of Gilbert.....892-0956
Maricopa County Highway Department.....262-3631

See Exhibit A that is attached to these Special Provisions.

SUBSECTION 108.7 - DETERMINATION AND EXTENSION OF CONTRACT TIME:

The Contractor shall be responsible for having taken steps reasonably necessary to ascertain the nature and location of the work and the conditions which can affect the work and the cost thereof. Failure to do so will not relieve bidders from responsibility for successfully performing the work without additional expense to the Flood Control District. The Flood Control District assumes no responsibility for understandings, representations, or predictions concerning conditions of the work area during the period of the contract.

If performance of all or any part of the work is suspended, delayed, or interrupted by weather conditions or by a rise in the water level causing unstable ground conditions, an extension of the period for contract performance equal to the lost days will be granted by the Flood Control District. No claim for additional costs incurred because of such delay will be allowed.

If the Contractor finds it impossible for reasons beyond his control to complete the work within the contract time as specified or as extended in accordance with the provisions of this subsection, he may, at any time prior to the expiration of the contract time as extended, make a written request to the Engineer for an extension of time setting forth therein the reasons which he believes will justify the granting of his request. If the Engineer determines that the Contractor has proceeded with such diligence as would normally have ensured completion within the contract time, and that the reasons stated to justify a time extension are valid, he may extend the time for completion in such amount as conditions justify. The extended time for completion shall then be in full force and effect,

the same as though it were the original time for completion.

SUBSECTION 108.9 - LIQUIDATED DAMAGES:

The amount of $350.00 per calendar day or the actual costs incurred by the District for Engineering and Inspection Services, whichever is less, will be added to the Daily Charges as indicated by TABLE 108 - LIQUIDATED DAMAGES and will be deducted from monies due or to become due to the Contractor for each and every calendar day that work shall remain uncompleted after the time specified for the completion of the work in the proposal, or as adjusted by the Engineer.

SECTION 206 - STRUCTURE EXCAVATION AND BACKFILL: Structure excavation consists of the removal of material for the construction of the bridge pile caps and substructure units in accordance with the plans and Section 206 of the Uniform Standard Specifications.

Structure backfill consists of furnishing, placing and compacting select material and other backfill material in accordance with the plans and Section 206 of the Uniform Standard Specifications, except that the backfill for the floodway bridge piers may be excavated material.

Select Material shall conform to the requirements of Section 702 of the Uniform Standard Specifications.

No separate payment will be made for structure excavation and backfill or select material used as backfill, as such, and the cost thereof shall be included in the price bid for the items to which they are appurtenant.

SECTION 301 - SUBGRADE PREPARATION: The work under this section consists of the preparation of the subgrades for the Bridge Approaches in accordance with the plans and Section 301 of the Uniform Standard Specifications.

Direct payment will not be made for structural excavation, waste, haul, overhaul, clearing, rolling or for the disposal of waste materials.

The earthwork quantities shown on the plans are approximate and were determined during the design process. They are included to aid the bidder in formulating his bid.

SECTION 310 - UNTREATED BASE: Select Material and Aggregate Base shall conform to the requirements of Section 702 of the Uniform Standard Specifications. Select Material and Aggregate Base shall be crushed in accordance with Section 702.2. Select Material shall be Type "A".

The Contractor will be required to furnish the Engineer certified weight tickets covering all of the Select Material and Aggregate Base placed on the project. Final pay quantities will be based upon the scale tickets accepted by the Engineer.

SECTION 315 - BITUMINOUS PRIME COAT: The bituminous materials shall be Grade MC-70 or MC-250 liquid asphalt as determined by the Engineer. Prime coat shall be applied to the total width of the Aggregate Base Material at the rate of 0.12 gallon per square yard unless otherwise specified by the Engineer.

SECTION 321 - ASPHALT CONCRETE: The bituminous material to be used shall be AR-4000 Paving Asphalt and shall comply with Sections 710 and 711 of the Uniform Standard Specifications or AC-30 and shall comply with ADOT Table 705-1 as modified on October 16, 1981.

The mineral aggregate shall meet the grading requirements within the range of the specified tolerances for Mix-Designation C-3/4 in accordance with Section 710 of the Uniform Standard Specifications and Maricopa County Highway Department Supplement to the Uniform Standard Specifications.

The work shall fully comply with Section 321 except that no mineral filler or blending sand will be required.

In addition to pugmill type mixing plants, Drum Dryer Mixers will be allowed in accordance with Standard Specifications 710.8. The moisture content of the bituminous mixture immediately behind the paver shall not exceed three percent. If a drum dryer is used, the type of oil shall be AR-8000 or AC-30 as specified above.

The proper proportioning of the material at the cold feed shall be determined by the Contractor and approved by the Engineer prior to the production of asphalt concrete. Production shall not commence until calibration tests indicate that an acceptable product can be obtained.

The correct proportions of each aggregate size introduced into the mixer shall be drawn from the storage bins by an approved type of continuous feeder which shall supply the correct amount of aggregate in proportion of each aggregate size can be separately adjusted. The continuous feeder for the aggregate may be mechanically or electrically activated.

The plant shall be equipped with a sampling device to take representative composite samples of the cold feed. If tests indicate non-compliance with specifications, operation shall cease until proper corrections have been made.

The production of the plant shall be governed by the rate required to obtain a thorough and uniform mixture of the materials. Mixing shall continue until the uniformity of coating, when tested in accordance with the requirements of AASHTO T195, is at least 95 percent.

SECTION 401 - TRAFFIC CONTROL: Traffic control shall conform to the applicable paragraphs of Section 401 of the MAG Uniform Standard Specifications with revisions, the County Supplement thereto and these Special Provisions.

A road closure is authorized. The Contractor shall provide continuous access to adjacent residences and to the RWCD Canal maintenance road. The Contractor shall provide all necessary traffic control devices and measures for this construction. Paint striping is not authorized on the permanent pavement which will remain after construction. The County shall stripe the finished road and bridges. All warning signs shall be standard 48 inches in size and mounted on channels or wire down.

Cost for providing and maintaining traffic control will not be paid for separately, but shall be incidental to the contract.

SECTION 415 - BRIDGE GUARD RAIL: The work under this section consists of constructing steel beam guard rail in accordance with the plans and Section 415 of the Uniform Standard Specifications, and MAG detail sheets 135-1 through 135-2.

SECTION 420 - CHAIN LINK FENCE: The work under this section consists of constructing chain link fence on the bridge decks in accordance with the plans and Section 420 of the Uniform Standard Specifications.

SECTION 501 - CONCRETE AND STEEL PILE FOUNDATIONS: The work under this section consists of constructing the concrete piles for the canal bridge and steel piles for the detour road bridge in accordance with the plans and the applicable provisions of Section 501 of the Uniform Standard Specifications.

Concrete and reinforcing steel will be paid for separately as indicated in Section 505 - Concrete Structures.

Payment for all other work under this section will be made at the unit price bid per linear foot for ITEM NO. 501-1 STEEL PILES FURNISHED, ITEM NO. 501-2 STEEL PILES DRIVING.

SECTION 502 - DRILLED SHAFT FOUNDATIONS:

Description: This item shall govern for the construction of foundations consisting of "reinforced concrete drilled shafts". Concrete shafts shall be placed in a drilled excavation. Foundations shall be constructed in accordance with this item and in conformance with the details and dimensions shown on the plans.

Materials: All concrete and materials shall be in accordance with the requirements of Section 505 of the Standard Specifications and the requirements herein. Concrete shall be f'c=4000 psi unless otherwise shown on the plans. The maximum size coarse aggregate shall be 1-1/2 inches for cased shafts. A retarder or water reducing agent will be required in all concrete when casing is used. Reinforcing steel shall conform to the requirements of Section 727 of the Standard Specification. The sizes and dimensions shall be as shown on the plans.

Construction Methods:

Excavation: The Contractor shall perform the excavation required for the shafts, through whatever materials encountered, to the dimensions and elevations shown on the plans or required by the site conditions.

Shaft alignment shall be within a tolerance of one inch per ten feet of depth. The axis of the shaft at the top elevation shall be within three inches of its plan location.

The plans indicate the expected depths and elevations where satisfactory bearing material will be encountered. This information will be used as a basis for the contract. If satisfactory material is not encountered at plan elevation, the footing may be raised or lowered as determined by the Engineer. Alteration of plan depth shall be made to satisfactorily comply with the design requirements. Casing will be required when necessary to prevent caving of the material or when necessary to exclude seepage water. Casing shall be metal of ample strength to withstand handling stresses, the pressure of concrete and of the surrounding earth or backfill materials, and shall be watertight. The outside diameter of casing shall not be less than the specified size of shaft; otherwise, the size of casing and the size of drilled excavation in which it is to be placed will be left to the discretion of the Contractor, except as noted below. No extra compensation will be allowed for concrete required to fill an oversize casing or oversize excavation.

Where caving conditions and/or excess groundwater is encountered, no further drilling will be allowed until a construction method is employed which will prevent excessive caving that will make the excavation appreciably larger than the size of casing to be used. Drilling with a vibrating casing or other method which will control the size of excavation, will be required.

If the excavation of the top of shaft is below ground level at the time of concrete placement, a casing from ground elevation to a point below the top of the shaft will be required to control caving of any material into the freshly placed concrete.

The cage shall be supported from the top by some positive method, to minimize its slumping downward during concrete placement and/or extraction of the casing. The support shall be concentric with the cage to prevent racking and distortion of the steel. A minimum of one-half of the vertical bars shall be supported.

In uncased shafts, concrete spacer blocks, or steel chairs shall be used at sufficient intervals to insure concentric spacing for the entire length of the cage. In cased shafts, concrete spacer blocks shall not be used. Metal "chair" type spacers or bent pieces of steel bars shall be placed at sufficient intervals around the steel cage to insure concentric spacing inside the casing.

Concrete: The work shall be performed in accordance with the provisions of the Standard Specifications and the requirements herein.

Concrete shall be placed as soon as possible after all excavation is complete and reinforcing steel placed and shall be of such workability that vibrating or rodding will not be required.

Concrete placing shall be continuous in the shaft to the construction joint indicated on the plans.

Concrete shall be placed through a suitable tube or tremie to prevent segregation of materials. The tube or tremie shall be made in sections to provide proper discharge and permit raising it as the placement progresses. A nonjointed pipe may be used if sufficient openings of the proper size are provided to allow for the flow of concrete into the shaft.

The elapsed time from the beginning of concrete placement in the cased portion of the shaft, until extraction of the casing is begun, shall not exceed one hour.

Where a cap or tie beam is required to be placed monolithically with the shaft, a time interval will be allowed for placing the required form and reinforcing after casing removal.

A riser block of equal diameter as the column end or a maximum height of 6 inches may be cast at the top of the completed shaft.

The top surface shall be cured and any construction joint area shall be treated as prescribed in the Standard Specification, Section 505.

The exposed portion of the drilled shaft shall be as shown on the drawing.

Formed Surface: Formed surfaces shall receive Class II finish.

Measurement: Acceptable drilled shafts (of the specified diameter) complete in place, will be measured by the linear foot. Shafts will be measured from elevations or dimension indicated on the plans, unless the Engineer directs otherwise to meet unusual conditions.

Payment: Drilled shafts will be paid for at the unit price bid per linear foot of drilled shaft of the specified diameter for ITEM NO. 502 - DRILLED SHAFT FOUNDATIONS. F'c=4000 psi concrete and reinforcing steel will be included in and paid for as items under Section 505 - CONCRETE STRUCTURES.

SECTION 505 - CONCRETE STRUCTURES: The work under this section consists of constructing, in place, the concrete portions of the bridges and approach slabs, in accordance with the plans and Section 505 of the Uniform Standard Specifications. Other methods, as described in 505.8 will be allowed in place of water curing.

The use of fly ash will be permitted in all concrete mixes, in accordance with subsection 725.2.1. Payment of all work under this section will be made at the unit price bid per cubic yard for ITEM NO. 505-1 CLASS AA f'c=4500 psi CONCRETE, ITEM NO. 505-2 CLASS AA f'c=4000 psi CONCRETE, ITEM NO. 505-3 CLASS A f'c=3000 psi CONCRETE, ITEM NO. 505-4 CLASS B f'c= 2500 psi CONCRETE and at the unit price bid per pound for ITEM NO. 505-5 REINFORCING STEEL.

SECTION 515 - STEEL STRUCTURES: The work under this section consists of placing the steel deck units for the Detour Bridge in accordance with the plans and the applicable provisions of Section 515 of the Uniform Standard Specifications.

Structural Calculations
GUADALUPE ROAD BRIDGE
OVER THE RWCD FLOODWAY

Bridge Consultant:	CANNON & ASSOCIATES, INC. CONSULTING ENGINEERS
Roadway Consultant:	THE WLB GROUP
Geotechnical Consultant:	SERGENT, HAUSKINS & BECKWITH

D E S I G N S U B M I T T A L

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CA Project No. 83-58
October 8, 1984

Sheet 1

Cannon & Associates, Inc.
Consulting Engineers
Tucson, Arizona

Basis For Design

Ref - 1) "Scope of Work" by Maricopa Co. Highway Dept
Rec'd Feb 24, 1984
2) "Std Spec. For Highway Bridges" by AASHTO, 1977, Updated
3) Technical Advisories - Referenced when used
4) Navdocks DM-7

Loading:

A) Design to Accomodate HS20-44 Loading
B) Calculations based on Service Loads & The working Stress Method
C) Thermal Range: Rise : 30°F (per AASHTO)
Fall : 40°F
Mean : 70°F
D) Seismic Zone: AASHTO Risk Zone 2
E) Stream Flow: V = 7.0 ft/s at d = 7.2', Q = 4900 cfs

Materials

A) Concrete

Superstructure	f'_c = 4500 psi
Substructure	f'_c = 4000 psi
All other Concrete	f'_c = 3000 psi

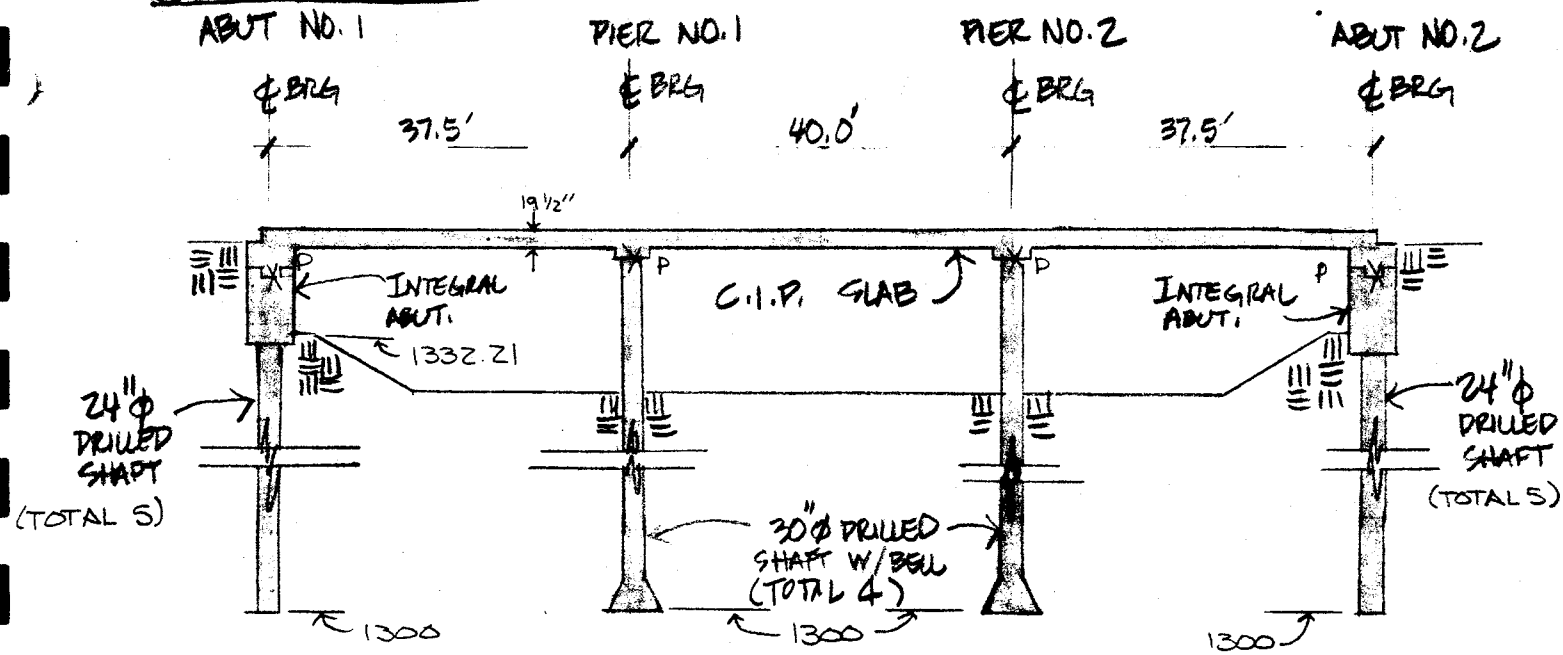
B) Steel

Bars #6 and smaller - Grade 40	f_s = 20,000 psi
Bars #7 and larger - Grade 60	f_s = 24,000 psi

Primary Assumptions

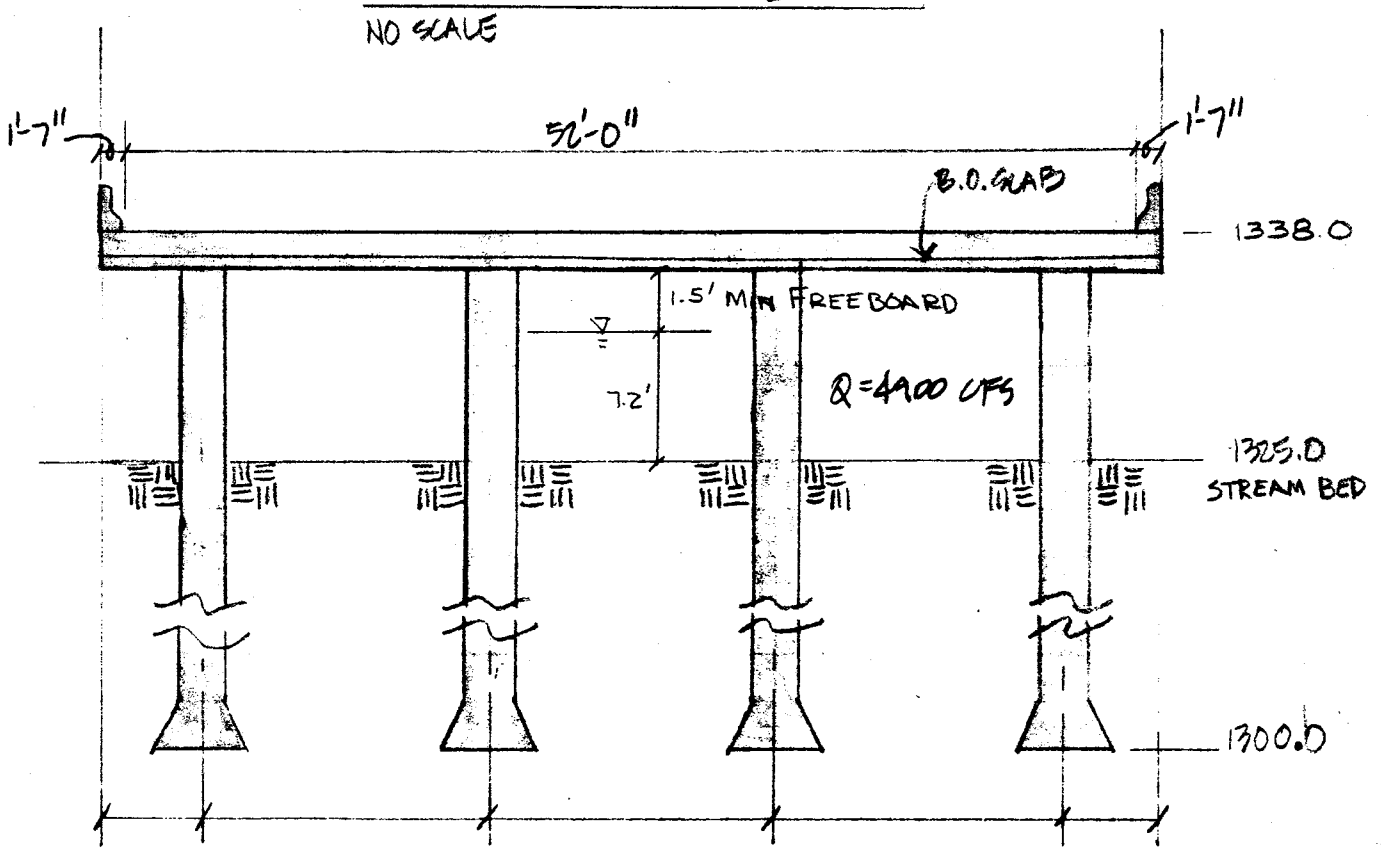
1) Pier Columns are pinned to girders - Slab Continuous
2) Integral Abutment - Abutment cap is pinned to caissons
3) Pier Design Configuration shown on following page
4) Scour Bed Elev. @ 1315.0
- Allows for 10' scour

CONFIGURATION



ELEVATION THRU ℄ CONST.

NO SCALE



PIER ELEVATION

NO SCALE

Cannon & Associates, Inc.
Consulting Engineers

project Guadalupe Br 83-58
designed by JS, RR
checked by MRE 10.8.84 RT
date 7/84 sh. 3 of

DECK DESIGN

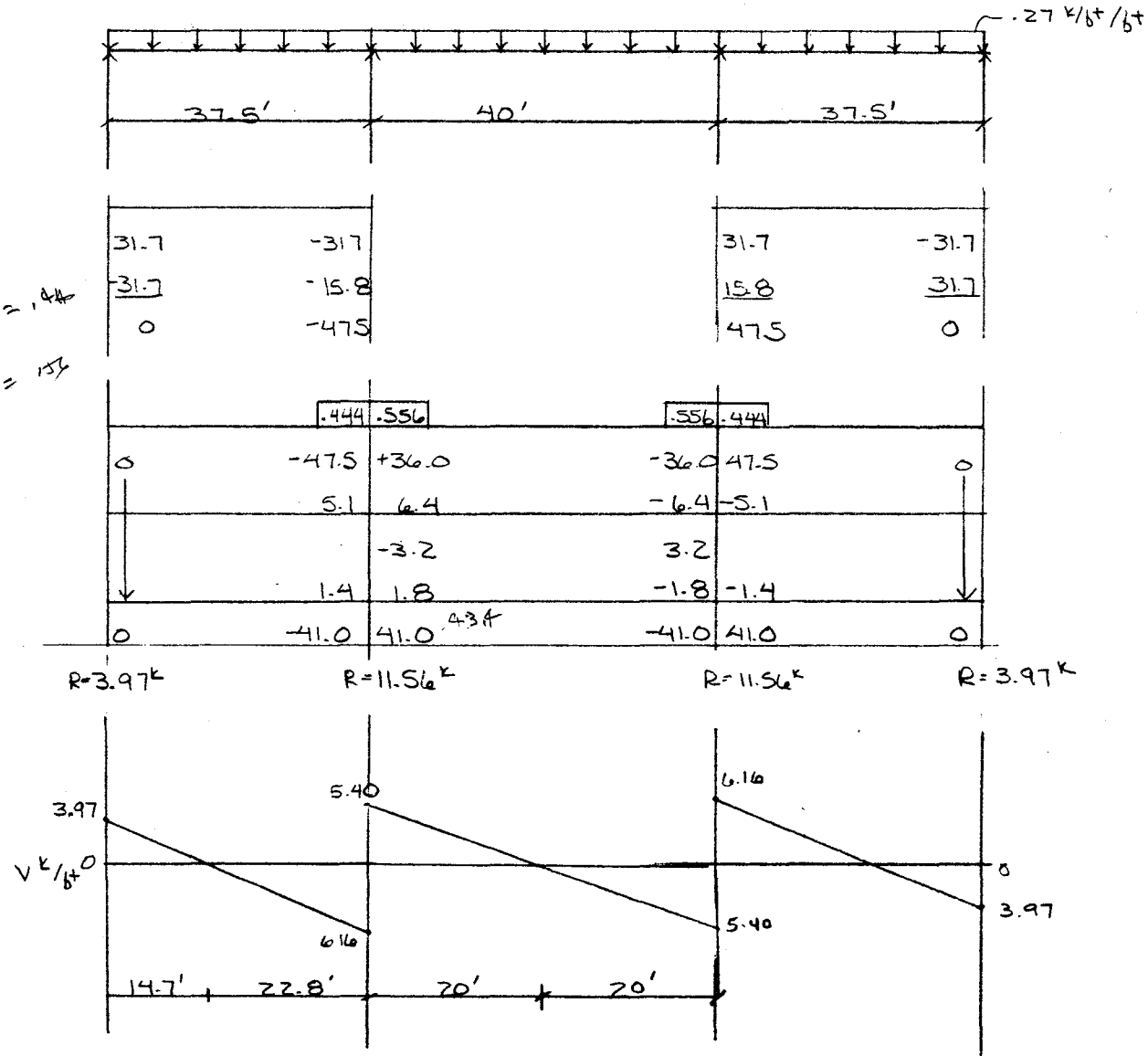
Slab Design

Dead Load: Wearing Surface = 0.025 ksf
Slab: $(19.5/12)(.150)$ = 0.244 ksf
0.269 ksf

$\frac{40+10}{30} = 1.67$
1.8

Note: Barriers Supported by edge beam

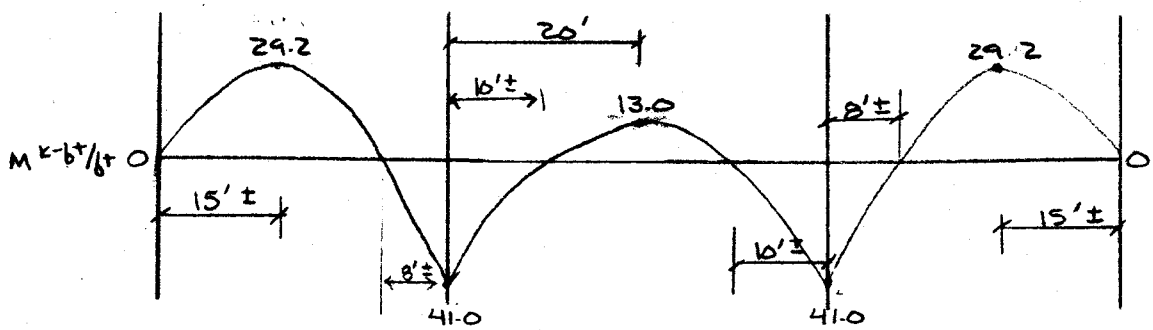
Assume superstructure is pinned to substructure, typ.



$\frac{3}{37.5} = .08 = .44$

$\frac{4}{40} = \frac{.10}{.18} = 15\%$

Dead Load Moment Diag:

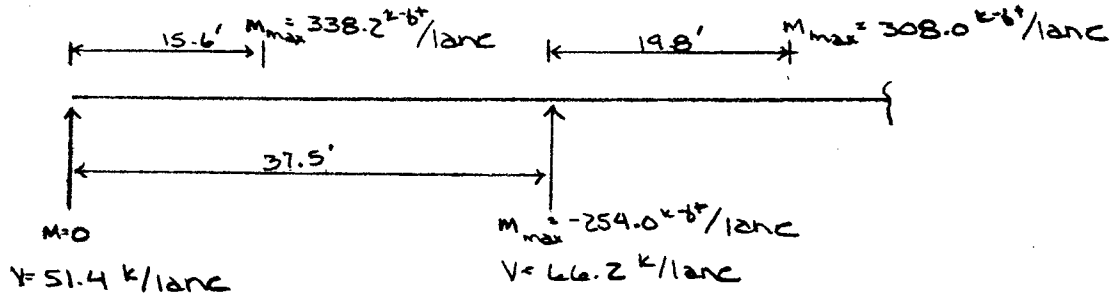


Live Load Moments

From 'Moments, Shears & Reactions For Continuous Highway Bridges'

$NL/L = 40/37.5 = 1.07$

Using Total Length = 120' & N = 1.1 (Table 3.1) - See Appendix



Impact Coefficient = 0.3

Distribution of Lane Loads: (AASHTO 1.3.2 (c))

$2E = 2(4+0.06(37.5)) = 12.5'$ (37.52' Span)

$= 2(4+0.06(40)) = 12.8'$ (40' Span)

DL+LL+I (Moments/ft width)

AB: DL = 29.2 k-ft

LL+I = 1.3(338.2/12.5) = 35.2 k-ft/ft D+L+I = 64.4 k-ft/ft

B: DL = -41.0 k-ft

LL+I = 1.3(-254.0/12.5) = -26.4 k-ft/ft D+L+I = -67.4 k-ft/ft

BC: DL = 13.0

LL+I = 1.3(308/12.8) = 31.3 k-ft/ft D+L+I = 44.3 k-ft/ft

d = 19½" Use spacing of bars = 6"

Span AB - Pos Reinforcement

M = 64.4 k-ft Try #10 @ 6" $A_s = 2(1.27) = 2.54$ in²

d = 19½" - 1" - 1.27/2 = 17.8" use d = 17"

$A_s/bd = \frac{2.54 \text{ in}^2}{(12")(17")} = 0.0124$ $n = \frac{29000}{3850} = 7.5$ use n = 8 ⇒ k = 0.357 j = 0.881

req'd $d = \sqrt{\frac{M}{\frac{1}{2} f_c k j b}} = \sqrt{\frac{64.4(12)}{\frac{1}{2}(1.8)(0.357)(.881)(12)}} = 15.1" < 17"$ O.K

$f_s = \frac{M}{A_s j d} = \frac{64.4(12)}{(2.54)(.881)(17)} = 20.3 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.357(20.3)}{8(1-0.357)} = 1.41 \text{ ksi} < 1.8 \text{ ksi}$

∴ O.K. <u>Use #10 @ 6"</u>

Interior Supports - Neg. Reinforcement

$M = 67.4^{k\text{-}ft}$ Try #10 @ 6" $A_s = 2.54 \text{ in}^2$ $d = 19\frac{1}{2} - 2\frac{1}{2} - 1.27/2 = 16.4''$ use $d = 16''$

$\frac{A_s}{bd} = \frac{2.54}{(12)(16.)} = 0.0132 \Rightarrow k = 0.366 ; j = 0.878$

req'd $d = \sqrt{\frac{(67.4)(12)}{0.5(1.8)(0.366)(0.878)(12)}} = 15.2'' < 16''$ o.k

$f_s = \frac{(67.4)(12)}{(2.54)(.878)(16)} = 22.7 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{0.366(22.7)}{8(1-0.366)} = 1.64 \text{ ksi} < 1.80 \text{ ksi}$ ∴ o.k.

Span BC - Pos Reinforcement

$M = 44.3^{k\text{-}ft}$ Try #8 @ 6" $A_s = 2(0.79) = 1.58 \text{ in}^2$ $d = 19\frac{1}{2} - 1 - 1.00/2 = 18''$

$A_s/bd = 1.58/(12)(18) = 0.0073$ $k = 0.288 ; j = 0.904$

$f_s = \frac{(44.3)(12)}{(1.58)(0.904)(18)} = 20.7 < 24.0 \text{ ksi}$

$f_c = \frac{0.288(20.7)}{8(1-0.288)} = 1.05 < 1.80 \text{ ksi}$ o.k.

Development Lengths: (AASHTO 1.5.14)

#10 bar: $l_d = \frac{0.04 A_b f_y}{\sqrt{f'_c}} = \frac{0.04(1.27)(60{,}000)}{\sqrt{4500}} = 45.4''$

Top Reinf.: $1.4(45.4'') = 63.6''$

Pos. Reinf.: $l_d = 0.8(45.4'') = 36.3''$

Neg. Reinf: $l_d = 0.8(63.6'') = 50.9''$

#8 bar: $l_d = \frac{0.04(0.79)(60{,}000)}{\sqrt{4500}} = 28.3''$

Pos. Reinf.: $l_d = 0.8(28.3) = 22.6''$

Development of Top & Bottom of Slab Reinforcement Plans:

Ref.: Bridge Planning and Design Manual, Vol III, pg 4-12 - See Appendix

AASHTO 1.5.13

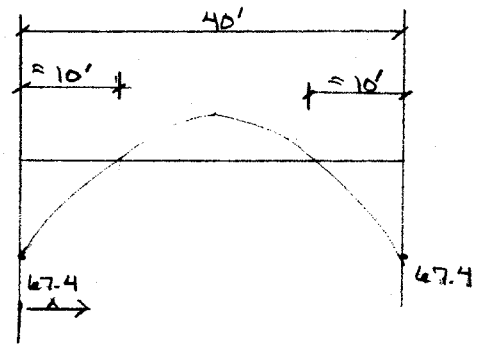
Top of Slab - Neg Reinforcement: (See Plan)

#10 bar $l_d = 50.9''$ $d = 16''$

$15 d_b = 15(1.27) = 19.05''$

$\ell/20 = 40'(12)/20 = 24''$ - Controls

$l_d + \ell/20 = 50.9'' + 24'' = 75'' = 6'-3''$

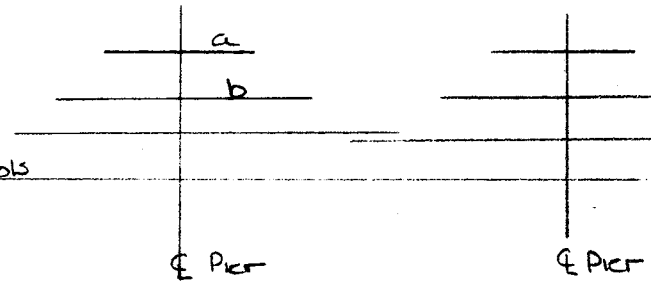


Bar a: $\ell = 2(6'-3'') = 12'-6''$

Theorectical cut-off pt. bar a:

@ $M = 2/3\,(67.4)\left(\frac{24}{22.7}\right) = 47.5$ k-ft

$x \approx 2.5'$



Bar b: $\ell = [2.5' + 50.9''/12]\,2 = 13.5'$

or

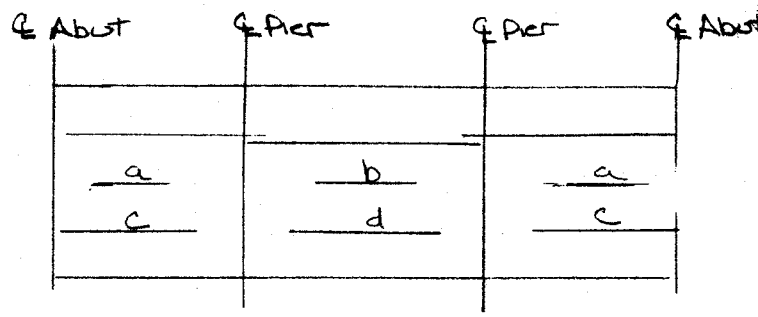
$\ell = 2[6.25' + 2'] = 16.5'$ - controls

Use $\ell = 19'-6''$

Bottom of Slab - Pos. Reinforcement: (See Plan)

#10 bar $l_d = 36.3''$ $\ell/20 = 37.5(12)/20 = 22.5''$ $36.3 + 22.5 = 58.8''$

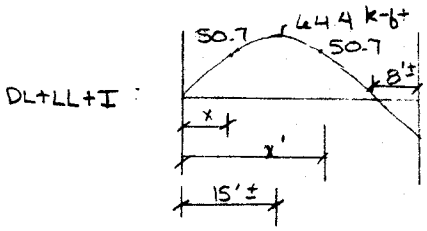
#8 bar $l_d = 22.6''$ $\ell/20 = 24''$ $22.6'' + 24'' = 46.6''$



Bars a & c (#10)

Theoretical cutoff pt - bar a

@ M = 2/3(64.4)(24.0/20.3) = 50.7 k-ft



x ≈ 8'

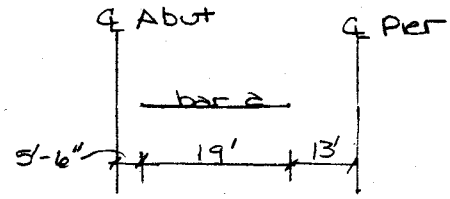
x' ≈ 22'

Length of bar a:

$\ell = 2(58.8'') = 117.6'' = 10'$

or $\ell = (22-8) + 2(22.5/12) = 17.75'$

use $\ell = 19'$



Bar b:

Theoretical cutoff point:

@ M = 1/3(64.4)(24.0/20.3) = 25.4 k-ft

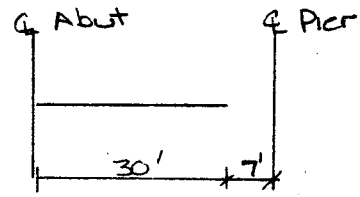
x ≈ 3.5'

x' ≈ 26.5'

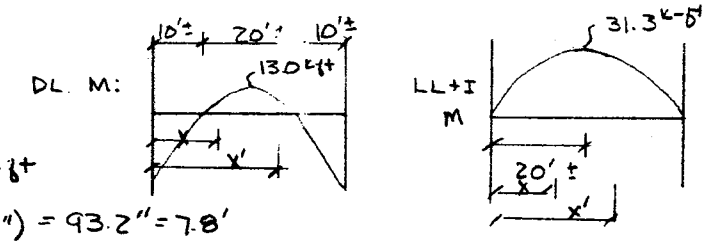
$\ell = (26.5 - 3.5) + 2(22.5/12) = 26.75'$

or $\ell = 19' + 2(36.3/12) = 25.05'$

use $\ell = 30'$



Bars b & d (#8)



Theoretical cutoff pt bar b

@ M = 2/3(44.3)(24/21) = 33.7 k-ft

x ≈ 12'

x' ≈ 28'

$\ell = 2(46.6'') = 93.2'' = 7.8'$

or $\ell = (28-12) + 2(24/12) = 20'$

use $\ell = 22'$

Bar d:

Theoretical cut-off point @ $M = 1/3(44.3)(24/21) = 16.9$ k-ft

$x \approx 6'$ $\quad \ell = 20' + 2(22.6/12) = 24'$

$x' \approx 34'$ $\quad \ell = (34-6) + 2(24/12) = 32'$ $\quad$ use $\ell = 32'$

Required Transverse Reinforcement - (AASHTO 1.3.2(E), 1.5.12)

Bottom of Slab:

Distribution Reinforcement $\% = \frac{100}{\sqrt{S}}$ $\quad$ 40' Span $\% = \frac{100}{\sqrt{40}} = 15.8\%$

37.5' Span $\% = \frac{100}{\sqrt{37.5}} = 16.3\%$

40' Span (#8 @ 6") $A_s = .158\left[\left(\frac{21\text{ ksi}}{24\text{ ksi}}\right)(1.58\text{ in}^2)\right] = 0.22\text{ in}^2$

use #5 @ 14" $A_s = 0.27\text{ in}^2$

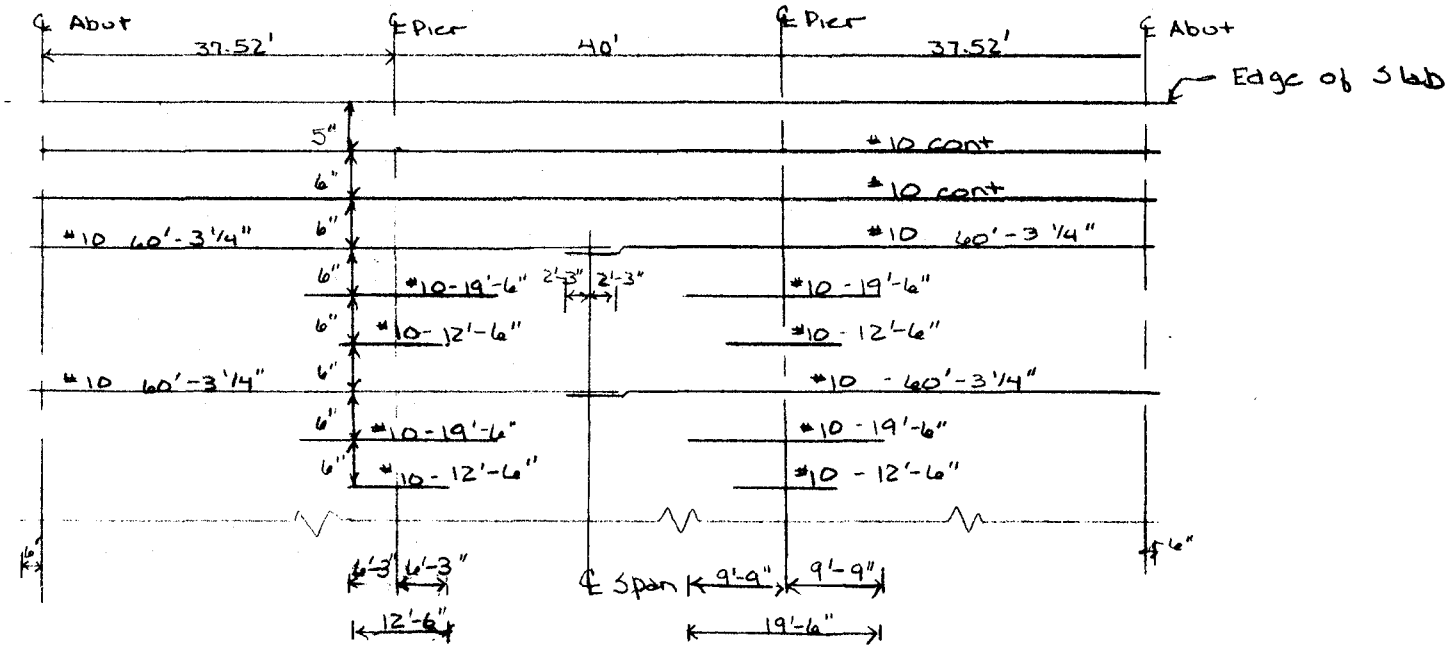
37.5' Span (#10 @ 6") $A_s = .143\left[\left(\frac{20.4}{24}\right)(2.54)\right] = 0.35\text{ in}^2$

use #5 @ 9" $A_s = 0.41\text{ in}^2$ ← controls

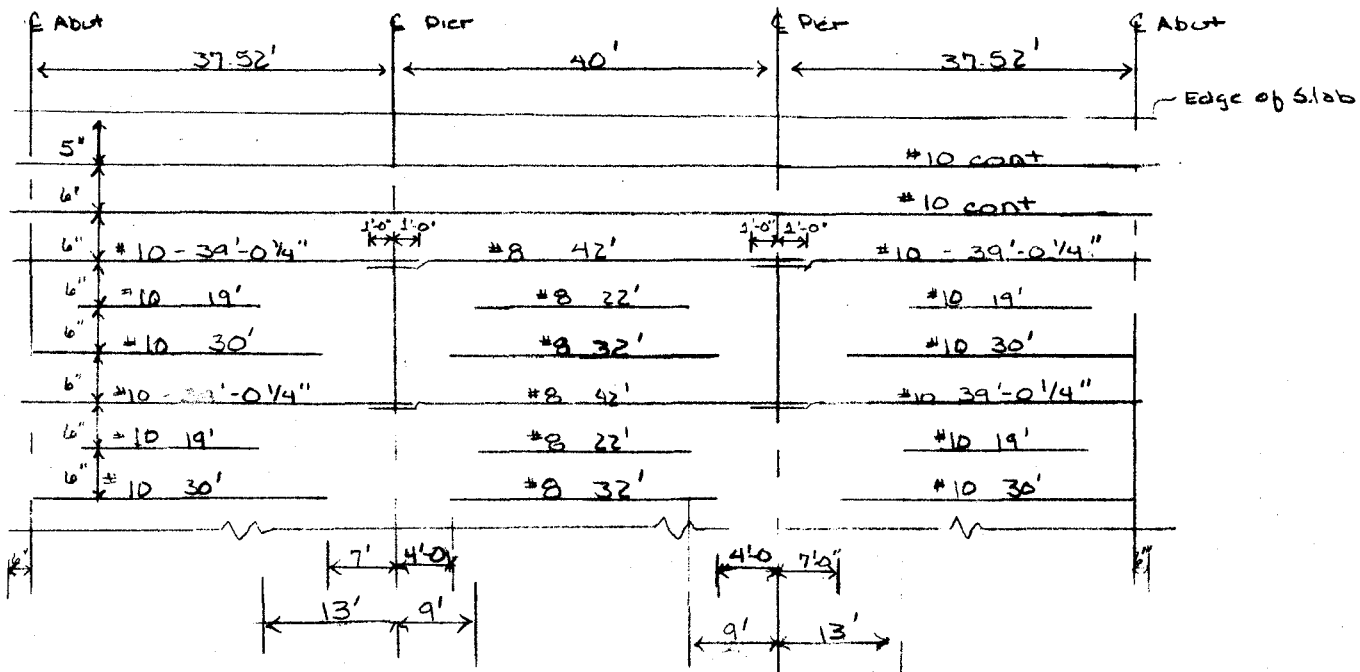
∴ use #5 @ 9"

Top of Slab:

Distribution Reinforcement - use #4 @ 18"



Reinforcement - Top of Slab



Reinforcement - Bottom of Slab

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date 7/16/84 sh. 12 of

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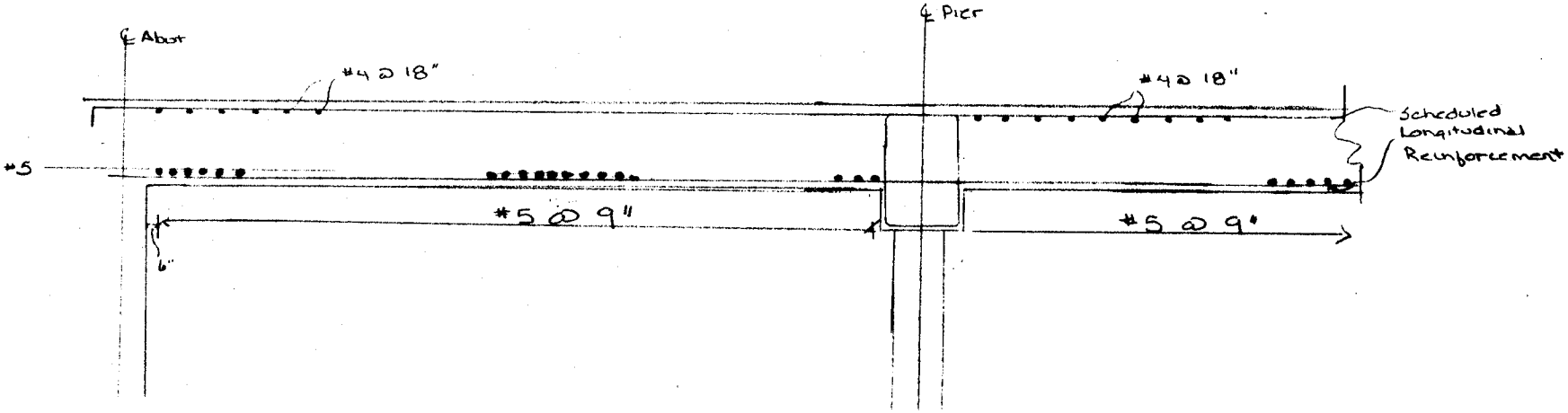

Transverse Deck Reinforcement

NTS

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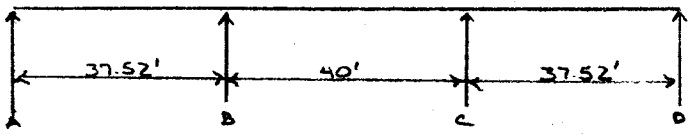
Edge Beam Design

width of beam = 1'-0" depth = 19½" + 10" = 29.5"



From Deck calculations:

DL Moments due to slab:

at ≈0.4 AB $M = (29.2\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = 26.5\ k\text{-}ft$

at B $M = (-41.0\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = -37.2\ k\text{-}ft$

at ≈0.5 BC $M = (13.0\ k\text{-}ft/ft)(1\ ft)\left(\frac{0.245}{0.270}\right) = 11.8\ k\text{-}ft$

DL Moments due to barrier:

barrier = 0.4 k/ft at ≈0.4 AB $M = \left(\frac{0.4}{0.27}\right)(29.2) = 43.2\ k\text{-}ft$

at B $M = \left(\frac{0.4}{0.27}\right)(-41.0) = -60.7\ k\text{-}ft$

at 0.5 BC $M = \left(\frac{0.4}{0.27}\right)(13.0) = 19.2\ k\text{-}ft$

Total D.L. Moments: at 0.4AB $M_D = 69.7\ k\text{-}ft$

at B $M_D = -97.9\ k\text{-}ft$

at 0.5BC $M_D = 31.0\ k\text{-}ft$

41.268

Live Load moment of 0.10 PS P = 16^k

Influence Coefficients from 'Moments, Shears & Reactions - -'

$NL/L = 40/37.52 = 1.07'$ Use Tables A3.0 & A3.1 - See Appendix

Max + mom @ 0.4 AB:

Table A 3.0 - Load @ 0.4AB Coeff = 0.2042

Table A 3.1 - Load @ 0.4AB Coeff = 0.2056

Coeff = 0.2042 + .7(.2056 - .2042) = .2052

$M_{0.4AB}$ = .2052(16)(37.52) = 123.2 $^{k-ft}$

0.1(123.2 $^{k-ft}$) = 12.3 $^{k-ft}$

Max - mom @ B

Table A 3.0 - Load @ 0.6 AB Coeff = -.1024

Table A 3.1 - Load @ 0.6 AB Coeff = -.0982

Coeff = -.0995

M = -.0995(16)(37.52) = -59.73

0.1(59.73) = -6.0 $^{k-ft}$

Max + mom @ 0.5 BC

Table A3.0 Load @ 0.5 BC Coeff = 0.1750

Table A3.1 Load @ 0.5 BC Coeff = 0.1894

Coeff = 0.1851

M = .1851(16)(40') = 118 $^{k-ft}$

0.1(118) = 11.8 $^{k-ft}$

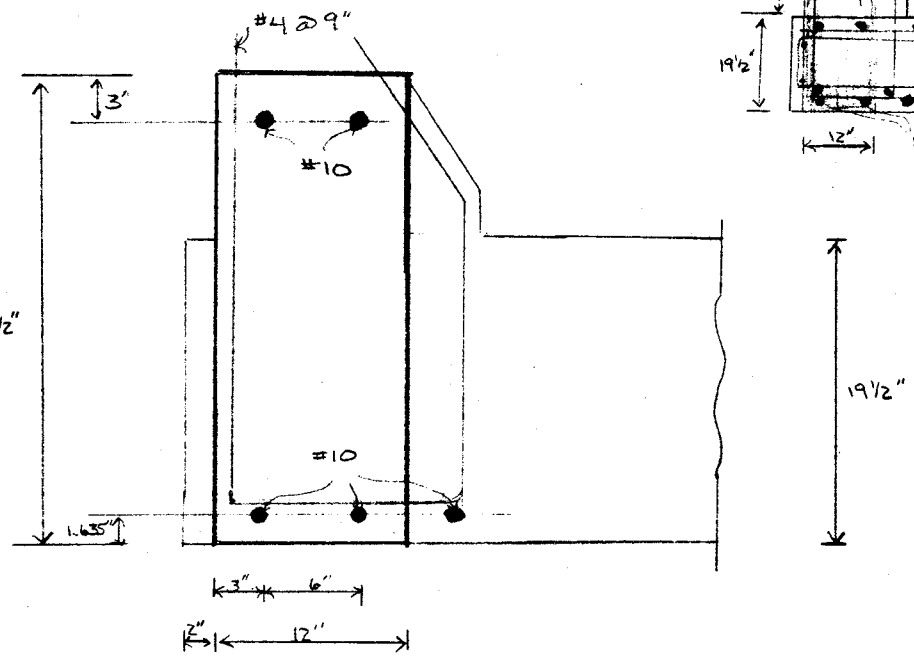
Total Moments:

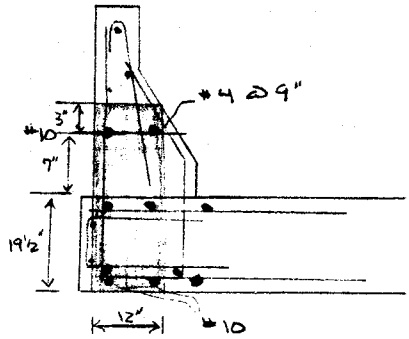
@ 0.4 AB M_T = 69.7 + 12.3 = 82.0 $^{k-ft}$

@ B M_T = -97.9 - 6.0 = -103.9 $^{k-ft}$

@ 0.5 BC M_T = 31.0 + 11.8 = 42.8 $^{k-ft}$

Edge Beam:





Design for $M = -103.9$ k-ft

$d = 29\frac{1}{2} - 3 = 26\frac{1}{2}''$ use $d = 26''$ $A_s = 2(1.27) = 2.54$ in^2

$n\frac{A_s}{bd} = 8\frac{2.54}{(12)(26)} = 0.0651$ $k = 0.301$ $j = 0.899$ req'd $d = \sqrt{\frac{103.9(12)}{\frac{1}{2}(1.8)(.301)(.899)(12)}} = 20.6''$

$20.6'' < 26''$ ✓

$f_s = \frac{103.9(12)}{(2.54)(.899)(26)} = 21.0$ ksi < 24.0 ksi

$f_c = \frac{0.301(21.0)}{8(1-0.301)} = 1.13$ ksi < 1.8 ksi ∴ Section o.k for moment

Check for shear:

$V_{max\ D.L.} = (6.16)(1')\left(\frac{0.245}{0.270}\right) + \left(\frac{0.4}{0.27}\right)(6.16) = 14.7^k$

from Deck Calc.

$V_{LL} = 0$ (Section Under Barrier)

$v = \frac{V}{bd} = \frac{14.7}{(12)(26\frac{1}{2}'')} = 0.046$ ksi

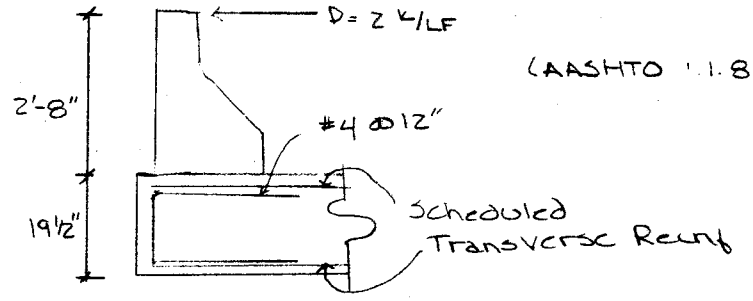
$V_c = 0.95\sqrt{4500} = 63.7$ psi > 46 psi > $63.7/2$ psi

AASHTO 1.5.10: req'd $A_v = \frac{50 b_w s}{f_y}$

for $s = 9''$ $A_v = \frac{50(12)(9)}{40{,}000} = 0.135$ in²

#4 bar $A_s = 0.20$ in² > 0.135 in² ∴ Section OK for shear

Check Resistance to Barrier Load



(AASHTO 1.1.8(A))

Deck: $M/ft = (2^k/LF)(2.667') = 5.3^{k\text{-}'}/LF$

Top Transverse Reinf: #4 @ 18" $A_s/LF = 0.13$ in² $d = 19\frac{1}{2}'' - 2\frac{1}{2}'' - 1.27 - .5/2 = 15.5''$

2'-0"

#4 @ 12" $A_s/LF = 0.20$ in² $d = 19\frac{1}{2} - 2\frac{1}{2} - 1.27 - .5 - .5/2 = 14.98''$

Use $A_s/LF = 0.20 + 0.13 = 0.33$ in² $d = 15''$

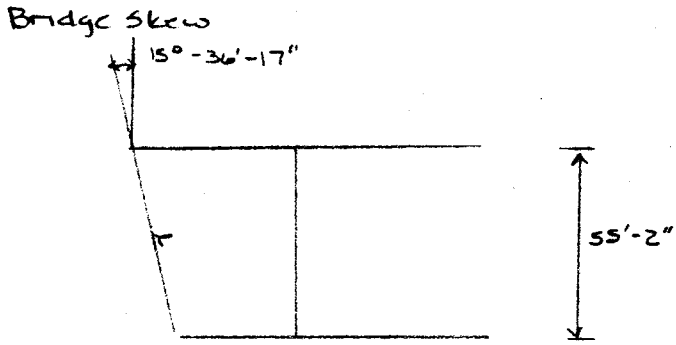
$n\frac{A_s}{bd} = 8\frac{0.33}{(12)(15)} = 0.0147$ $k = 0.157$ $j = 0.947$

$f_s = \frac{M}{A_s j d} = \frac{5.3(12)}{0.33(0.947)(15)} = 13.6$ ksi < 20 ksi

$f_c = \frac{0.157(13.6)}{8(1-0.157)} = 0.32$ ksi < 1.8 ksi ∴ O.K.

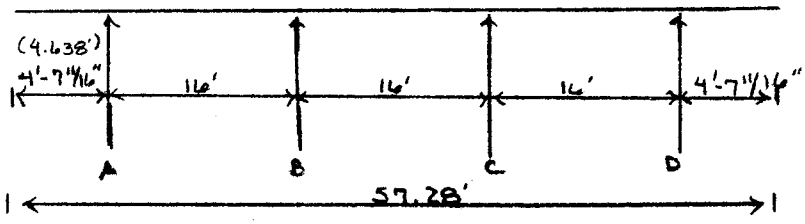
Pier Design

Pier Cap Design



Length of Pier Girder = 57.28'

Place piers 16' apart



From Slab Calculations:

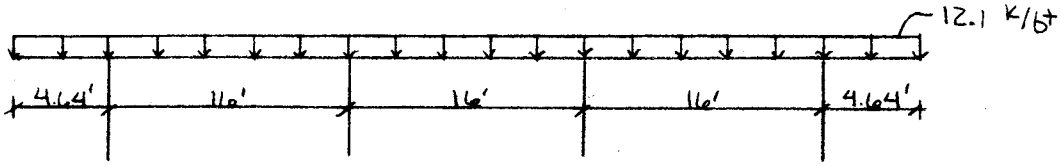
$R_{DL} = 11.56$ k/ft Due to skew $R_{DL} = 0.963(11.56) = 11.13$ k/ft

Use pier girder $h = 7\frac{1}{2}''$ $w = 4.0'$

Load from barrier = 0.4 k/ft
from slab calc. concentrated Load = $\frac{0.4}{0.27}(11.56) = 17.12$ k

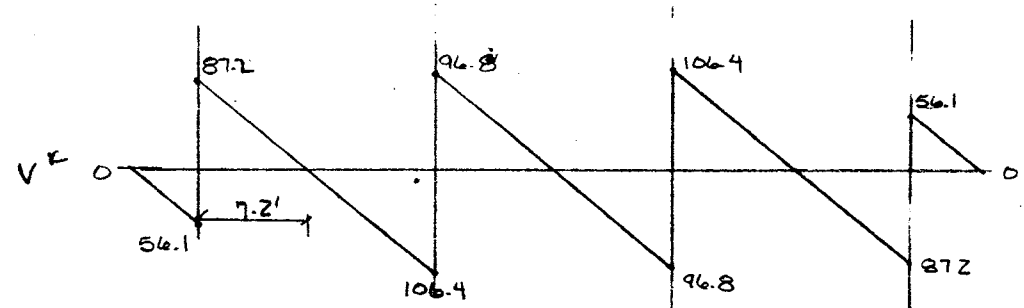
$w_{DL} = 11.13 + 2(17.12)/57.3' + (.150\,k/ft^3)(7.5/12)(4') = 12.1$ k/ft

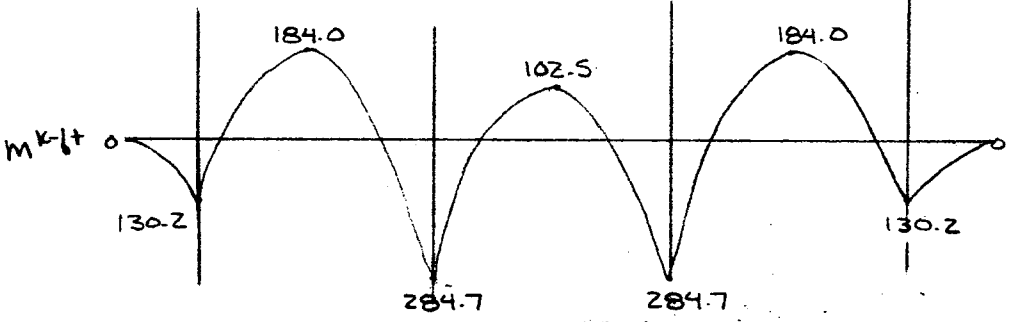
DL Mom Distr.:



130.2	-258.1	258.1		-258.1	258.1	-130.2
	127.9	64.0		-64.0	-127.9	
	-130.2	322.1		322.1	130.2	

		3/7	4/7	4/7	3/7	
	-130.2	322.1	-258.1	258.1	-322.1	130.2
		-27.4	-36.6	36.6	27.4	
			18.3	-18.3		
		-7.8	-10.5	10.5	7.8	
			5.2	-5.2		
		-2.2	-3.0	3.0	2.2	
Σ	-130.2	284.7	-284.7	284.7	-284.7	130.2
	R = 143.3k		R = 203.2k	R = 203.2k		R = 143.3k





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Live Load -

Ref: 'Moments, Shears and Reactions for Continuous Highway Bridges'

Table 3.1 Interpolate for "L" = 37.5 - (See Appendix)

Max LL Reaction = 65.8 k/lane

No of lanes = 52/12 = 4.33 Use 4 lane (AASHTO 1.2.6)

$w_{LL} = 65.8(4)/(52'/\cos 15.60) = 4.88$ k/ft use $w_{LL} = 5.0$ k/ft

$+M = \frac{5.0}{12.1}(184) = 76.0$ k-ft

$-M = \frac{5.0}{12.1}(285) = 118$ k-ft

D+L+I:

$+M = 184 + 1.3(76) = 283$ k-ft

$-M = 285 + 1.3(118) = 438$ k-ft

Top Reinf: Use 10 #10

$-M = 438$ k-ft Try 10 #10 $A_s = 10(1.27) = 12.7$ in^2

$d = 27'' - 2\frac{1}{2}'' - 1.27'' - 1.27/2 = 22.6''$ use $d = 22''$

$n\frac{A_s}{bd} = 8\frac{12.7}{(48)(22)} = 0.096$ $k = 0.352$ $j = 0.882$

Req'd $d = \sqrt{\frac{438(12)}{\frac{1}{2}(1.8)(.352)(.882)(48)}} = 19.8'' < 22''$ ✓

$f_s = \frac{438(12)}{12.7(.882)(22)} = 21.3 < 24.0$ ksi

$f_c = \frac{0.352(21.3)}{8(1-.352)} = 1.45 < 1.80$ ksi ∴ o.k.

Bottom Reinf Use 10 #8

$+M = 283^{k\text{-}ft}$ Try 10 #8 $A_s = 10(0.79) = 7.9 \text{ in}^2$

$d = 27'' - 1\tfrac{1}{2}'' - .625'' - 1.0/2 = 24.4''$ Use $d = 24''$

$n \frac{A_s}{bd} = 8 \frac{7.9}{(48)(24)} = 0.0549$ $k = 0.281$; $j = 0.906$

$f_s = \frac{283(12)}{7.9(.906)(24)} = 19.8 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{0.281(19.8)}{8(1-.281)} = 1.0 \text{ ksi} < 1.8 \text{ ksi}$ ∴ O.K.

Shear Reinforcement:

@ d (22") from support: $V_D = 106.4^k - 12.1(22/12) = 84.2^k$

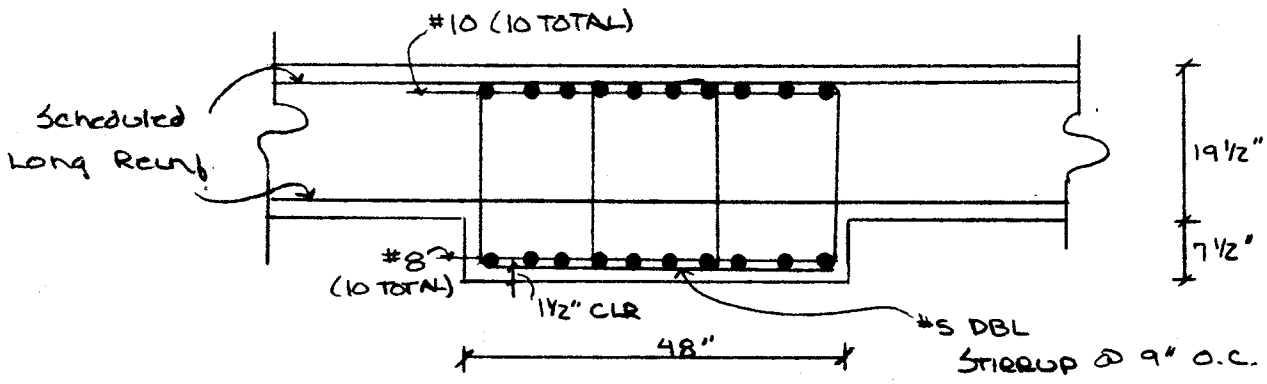
$V_L = 5.0^k/ft\,(14/2 - 22/12) = 30.8^k$

$V_{D+L+I} = 84.2^k + 1.3(30.8^k) = 125^k$

$v = V/bd = \frac{125^k}{(48)(22)} = 0.118 \text{ ksi}$ $v_c = 0.95\sqrt{4500} = 64 \text{ ksi}$

#5 DBL Stirrups: $A_v = 4(0.31) = 1.24 \text{ in}^2$

Req'd $s = \frac{1.24(20{,}000)}{(118-64)(48)} = 9.6''$ Use $s = 9''$



Pier Column

- Lateral Loading

Analysis Using NAVDOCKS DM-7 - Lateral Loads on Piles - See Appendix

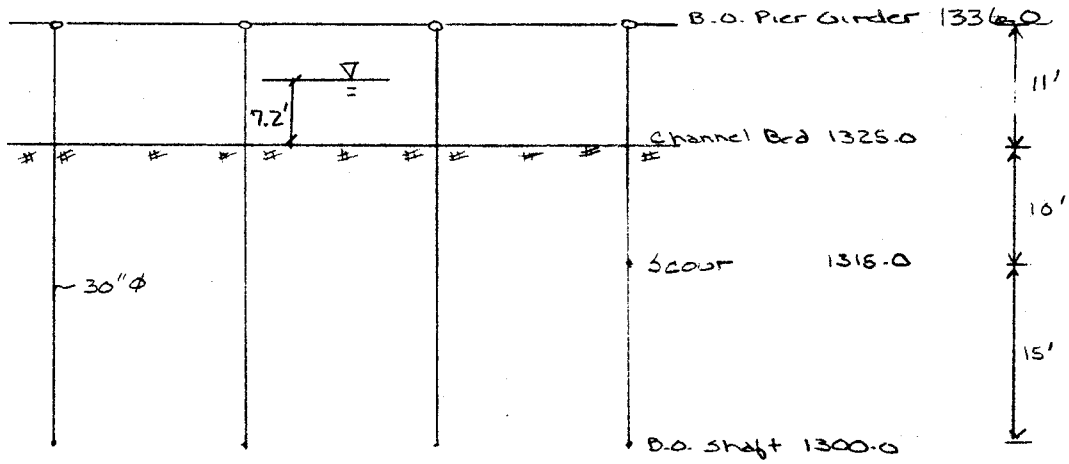
From Soil Report:

Coeff. of var. of lateral subgrade reaction $F = 60\ \#/in^3$

$F = 30\ \#/in^3$ w/ EQ

- Flexible Cap, Elevated Position Case

Transverse Model:



Stream Flow Forces:

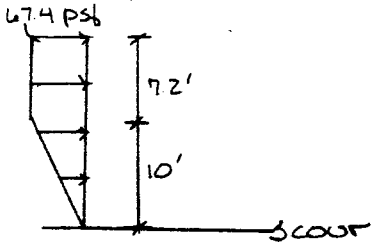
(AASHTO 1.2.17) $P = KV^2$

$V = 7.0$ ft/s $K = 1.375$ (assuming flat face from debris)

$P = 1.375(7.0)^2 = 67.4$ psf

water level @ 1332.2'

Assumed SF loading:



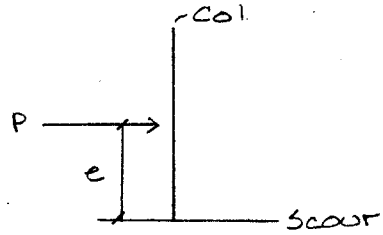
Stream Flow Forces (cont)

@ Debris laden pier effective col. width = 6.5'
(Per Corp of Engrs Recommendation)

$P = (67.4)(7.2)(6.5) + \frac{1}{2}(67.4)(10)(6.5)$

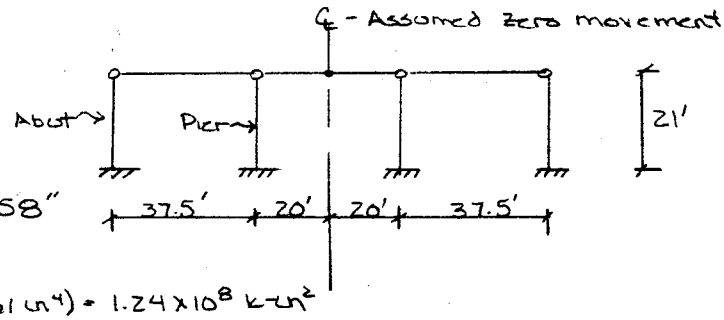
$= 3154^{\#} + 2190^{\#} = 5344^{\#}$

$$e = \frac{3154(10 + 7.2/2) + 2190(2(10)/3)}{5344} = 10.76'$$



Temperature Forces

Longitudinal Direction:

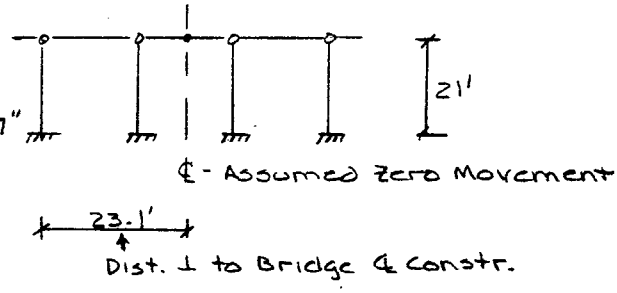


$\Delta_{pier} = (0.000006)(20')(12)(40°) = 0.058''$

$P = \frac{3EI\Delta}{L^3}$ $\quad EI = 57\sqrt{3000}\,(39{,}761\ in^4) = 1.24 \times 10^8\ k\text{-}in^2$

$P = \frac{3(1.24 \times 10^8)(0.058'')}{(21 \times 12)^3} = 1.35^k$ @ $e = 21'$

Transverse Direction:

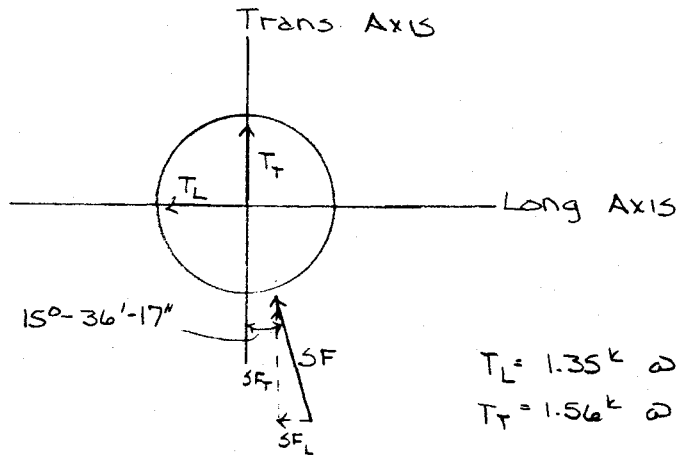


$\Delta_{ext\ pier} = (0.000006)(23.1')(12)(40) = 0.067''$

$P = \frac{3(1.24 \times 10^8)(.067)}{(21 \times 12)^3} = 1.56^k$

@ $e = 21'$

Load Case IV - Stream Flow & Temp.



$T_L = 1.35^k$ @ 21'

$T_T = 1.56^k$ @ 21'

$SF = 5.35^k$ $\quad SF_L = 5.35 \sin 15.6° = 1.44^k$, $SF_T = 5.35 \cos 15.6° = 5.15^k$ } @ 10.8'

$P_L = 1.35^k + 1.44^k = 2.79^k$ @ $\dfrac{1.35(21) + 1.44(10.8)}{2.79} = 15.7'$

$P_T = 1.56^k + 5.15^k = 6.71^k$ @ $\dfrac{1.56(21) + 5.15(10.8)}{6.71} = 13.2'$

$P = ((2.79)^2 + (6.71)^2)^{1/2} = 7.27^k$ @ $\dfrac{2.79(15.7) + 6.71(13.2)}{(2.79 + 6.71)} = 13.9'$

M @ scour $= 7.27^k (13.9') = 101.^{k\text{-}ft}$

Ref: NAVDOCKS DM-7 - See Appendix

$EI = 1.24 \times 10^8$ k-in^2 $\quad f = .06$ k/in^3 $\quad T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{1.24 \times 10^8}{.06}\right)^{1/5} = 73''$

$L = 15'$ $\quad L/T = 15(12)/73 = 2.5$

Max Moment: (1982 DM-7, Figure 11 - See Appendix)

@ $z/T = 0$ $\quad M_m = 1.0(101) = 101^{k\text{-}ft}$

$M_p = 0.0$

@ $z/T = 1.0$ $\quad M_m = 0.75(101) = 76^{k\text{-}ft}$

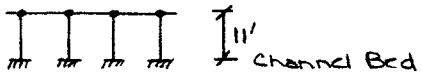
$M_p = 0.60(7.27)(73/12) = 26^{k\text{-}ft}$

$102^{k\text{-}ft}$ - controls

Earthquake Forces: (AASHTO 1.2.20)

EQ = CFW

F = 1.0



W = Dead Load:

Wearing Surface	$= (.025)(52)(115)$	$= 150^k$
Barriers	$= (.4^k/ft)(115)(2)$	$= 92^k$
Slab	$= (.150)(19.5/12)(115)(55.2)$	$= 1547^k$
Haunches	$= 2(.150)(4')(7.5/12)(57.3')$	$= 43^k$
Columns (to fun. grade)	$= 8(.150)(11)(\pi(1.25)^2)$	$= 65^k$
Abut. Diaphragm	$= 2(.150)(2.5')(3')(57.3')$	$= 129^k$
		2026^k

Determine P for structure $\quad P = P_{piers} + P_{abuts}$

$$P_{piers} = \frac{3EI\Delta}{L^3} \times 4 \text{ cols.} \times 2 \text{ piers} \qquad \Delta = 1'' \qquad EI = 1.24 \times 10^8 \ k\text{-}in^2$$

$$P_{piers} = \frac{3(1.24 \times 10^8)(1'')}{(11 \times 12)^3} \times 8 = 1294^k$$

Determine P_{abuts} using DM-7

$E = 3122$ ksi $\quad I = \pi(12'')^4/4 = 16286 \text{ in}^4 \quad EI = 5.08 \times 10^7 \text{ k-in}^2$

$f = 0.03 \text{ k/in}^3 \quad T = \left(\frac{5.08 \times 10^7}{.03}\right)^{1/5} = 70''$

$L = 1328' - 10' - 1300' = 28' \quad L/T = 28(12)/70 = 4.8$

$P_{abuts} = \frac{\Delta EI}{F_\Delta T^3} \times 5 \text{ shafts} \times 2 \text{ abuts} \quad \Delta = 1'' \quad @ \; z/T = 0 \quad F_\Delta = 2.3$

$P_{abuts} = \frac{1(5.08 \times 10^7)}{2.3\,(70)^3} \times 10 = 644^k$

$\therefore P = 1294^k + 644^k = 1938^k$

Determine EQ force / pier col.

Assumptions:

- Pier Columns Resist Entire Seismic Force
- Seismic Force Is Equally Distributed Between Columns
- C.G. of Structure @ B.O. Deck

$T = 0.32\sqrt{W/P} = 0.32\sqrt{2026/1938} = 0.33 \text{ sec} \rightarrow C = 0.08$ (AASHTO Fig. 1.2.20D)

$EQ = 0.08(1.0)(2026^k) = 162^k$

$EQ/_{col} = 162^k/8 = 20.2^k$

$M_{EQ} @ \text{Invert} = 20.2^k(11') = 222^{k\text{-}'}$

Determine Max M_{EQ} using DM-7 - See Appendix

$L = 25' \quad f = .03 \text{ k/in}^3 \quad T = \left(\frac{1.24 \times 10^8}{.03}\right)^{1/5} = 84'' \quad L/T = 25(12)/84 = 3.6$

$@ \; z/T = 0 \quad M_m = 1.0(222) = 222 \quad M_p = 0.0$

$@ \; z/T = 1.2 \quad M_m = 0.76(222) = 169^{k\text{-}'}$

$M_p = 0.73(20.2)(84/12) = 103^{k\text{-}'}$

Controls $\rightarrow 272^{k\text{-}'}$

Load Cases

Case IV: $1.0(D+L+SF+T) \leq 1.25$ - Stream Flow & Temperature

$M_{CASE\ IV} = 102^{k\text{-}ft}/1.25 = 81.6^{k\text{-}ft}$

$P_{CASE\ IV} = P_D + P_L/1.25$ (Ext. Col controls - Neglect wt. of col. below mud line)

$P_D = 143^k + 16^k = 159^k$

$P_L = 54^k$

$P_{CASE\ IV} = (159^k + 54^k)/1.25 = 170^k$

Case VII: $1.0(D+SF+EQ) \leq 1.33$

Assume EQ & SF Do Not Occur Simultaneously
- EQ Controls

$M_{CASE\ VII} = 272/1.33 = 205^{k\text{-}ft}$

$P_D = 143^k + 8^k = 151^k$

$P_{CASE\ VII} = 151/1.33 = 114^k$

∴ Load Case VII will control design

AASHTO 1.5.28, 1.5.34

Use $l_u = 11'$ (Streambed to b.o girder) per USDOT

$K = 2.1$ (fixed @ bott., pinned @ top)

$r = 0.25(30) = 7.5'$

$Kl_u/r = 2.1(11)(12)/7.5 = 37$

$\phi = 1.0$; $\beta = 0.0$ (No DL Mom transferred to columns); $C_m = 1.0$

$$EI = \left(\frac{E_c I_g}{2.5}\right)/1+\beta = \left(\frac{1.24\times10^8}{2.5}\right)/1+0 = 4.96\times10^7 \text{ k-in}^2$$

$$P_c = \frac{\pi^2(4.96\times10^7)}{((2.1)(11)(12))^2} = 6370^k$$

$$\delta = \frac{C_m}{1-\frac{P_u}{\phi P_c}} \qquad P_u = 2.5P \text{ (AASHTO 1.5.28)}$$

$$\delta = \frac{1.0}{1-\frac{2.5(114)}{6370}} = 1.05 \qquad M_c = \delta M = 1.05\,(205) = 215^{k-ft}$$

Design Loads: $M = 215/0.35 = 614^{k-ft}$

$P = 114/0.35 = 326^k$

$f_y = 60{,}000$ psi
$f'_c = 3{,}000$ psi
$D = 30''$
Use 12 #11 $A_{s1} = 1.56$
$N = 12$

Clear spacing of verts
$= 2(10.5'')\sin 30°/2 - 1.41'' = 4.02''$ O.K.

Use 1/2" Ø Spiral Ties @ 6" pitch

Note: used $f'_c = 4000$ psi,
Redesign Not Req'd

```
^^^^^^^^^^^^^^^^
CIRCON
vvvvvvvvvvvvvvvv

                    *P*
FY=60,000.00
Fc=3,000.00
D=30.00
D1=4.25
AS1=1.56
N=12.00
P=326.00
MX=614.00
MY=1.00E-8

REBAR COORDINATES
Rb=10.75 ∠=0.00
Rb=10.75 ∠=30.00
Rb=10.75 ∠=60.00
Rb=10.75 ∠=90.00
Rb=10.75 ∠=120.00
Rb=10.75 ∠=150.00
Rb=10.75 ∠=180.00
Rb=10.75 ∠=210.00
Rb=10.75 ∠=240.00
Rb=10.75 ∠=270.00
Rb=10.75 ∠=300.00
Rb=10.75 ∠=330.00

<.8>PMAX=1,611.65

ebR=15.47
MbR=8,083.27
PbR=522.67
cbR=15.24
B∠=9.33E-10

P=325.02
MX=666.78
MY=1.09E-8
cR=13.30
ecR=2.05
```

$> 614^{1k}$

O.K.

Pier- Broms Analysis (Check Lateral Load Capability)

Broms Analysis - See Appendix

Per Soil Report - Need to Consider 2 cases:

1) Cohesive case: $k_h D = 400^{\#}/in^2$ ($200^{\#}/in^2$ for seismic)

2) Cohesionless case: $n_h = 60^{\#}/in^3$ ($30^{\#}/in^3$ for seismic)

For ultimate lateral resistance: $\phi = 30°$, $C_u = 900$ psf

Passive Earth Pressure Coeff.: $K_p = 3.0$

$\gamma = 120^{\#}/ft^3$

Concrete Pier: $D = 30''$ $I = 39{,}761\ in^4$

$E = 3122$ ksi (3000 psi conc)

For Cohesive Case:

$$\beta = \sqrt[4]{\frac{k_h D}{4EI}} = \sqrt[4]{\frac{400}{4(3{,}122000)(39{,}761)}} = 0.0053 \quad (0.0045 \text{ for seismic})$$

L- Earthquake = 25' L- Stream Flow = 15'

SF: $\beta L = .0053(15)(12) = 0.95 < 2.25$

EQ: $\beta L = .0045(25)(12) = 1.35 < 2.25$ ∴ Rigid Pile Action

Earthquake: L = 25'
P = 20k
e = 11'

$e/D = 11/2.5 = 4.4$

$\frac{P}{C_u D^2} = \frac{20000}{(900\ psf)(2.5)^2} = 3.56$

$L/D = 5.0\pm$ $L = 5.0(2.5) = 12.5' < 25'$

$FS = 25/12.5 \times 1.33 = 2.66$

Stream Flow: Temp: L = 15'
P = 7.3k
e = 13.9'

$e/D = 13.9/2.5 = 5.56$

$\frac{P}{C_u D^2} = \frac{7300}{900(2.5)^2} = 1.30$

$L/D = 4\pm$ $L = 4.0(2.5) = 10 < 15'$

$FS = 15/10 \times 1.25 = 1.88$

Broms Analysis of Piers - Cont

For Cohesionless Case

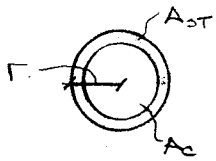
$$\eta = \sqrt[5]{\frac{\eta_h}{EI}} = \sqrt[5]{\frac{60\ \#/in^3}{(3{,}140{,}000\ psi)(39{,}761\ in^4)}} = 0.014 \quad (0.012\ \text{Seismic})$$

SF: $\eta L = 0.014(15)(12) = 2.52 > 2.0$

EQ: $\eta L = 0.012(25)(12) = 3.60 > 2.0$ $\therefore$ Flexible Pile Action

Conservatively use transformed uncracked section to calc. section properties:

Transformed section:



$A_s = 12(1.56) = 18.7\ in^2$

$A_c = \pi(15)^2 - 18.7 = 688\ in^2$

$n = \frac{29{,}000}{3{,}122} = 9.3$ $\quad A_{ST} = 9.3(18.7) = 174\ in^2$

$A_T = 688 + 174 = 862\ in^2$ $\quad r = \sqrt{862/\pi} = 16.6''$

$I = \pi(16.6)^4/4 = 59{,}638\ in^4$

$M_r = \frac{1200(59{,}638)}{16.6} = 4311\ k\text{-}in = 359\ k\text{-}ft$

$M_r / D^4 \gamma K_p = 359/(2.5^4)(.120)(3.0) = 25.5$

From Design Charts - Ultimate Analysis of Flexible Piles, Broms 'Design of Laterally Loaded Piles,' ASCE Proc 4342 May 1965

Stream Flow: Temp. $e/D = 5.56$; $P/K_p D^3 \gamma = 4.0 \pm$ $P_{allow} = 4.0(3.0)(2.5)^3(.120) = 22.5^k$

$FS = 22.5/7.3 \times 1.25 = 3.85$

EQ: $e/D = 4.4$ $\quad P/K_p D^3 \gamma = 4.5 \pm$ $P_{allow} = 4.5(3.0)(2.5)^3(.120) = 25.3^k$

$FS = 25.3/20 \times 1.33 = 1.68$

$\therefore$ Pile OK for lateral load

Pier Top @ Elev. 1300.0'

Axial Load = 203^k + 80^k = 283^k

From Fig 4 - Sergent, Hauskins & Beckwith Soil Report (attached)

For 5.0' Bell Diam @ 1300' Safe Capacity = 325^k > 283^k

Use 30" Ø Piers w/ 12 #11 Verts & 1/2" Ø Spiral Ties @ 6" o.c. w/ 5.0' Ø Bell & pier top elev. @ 1300'

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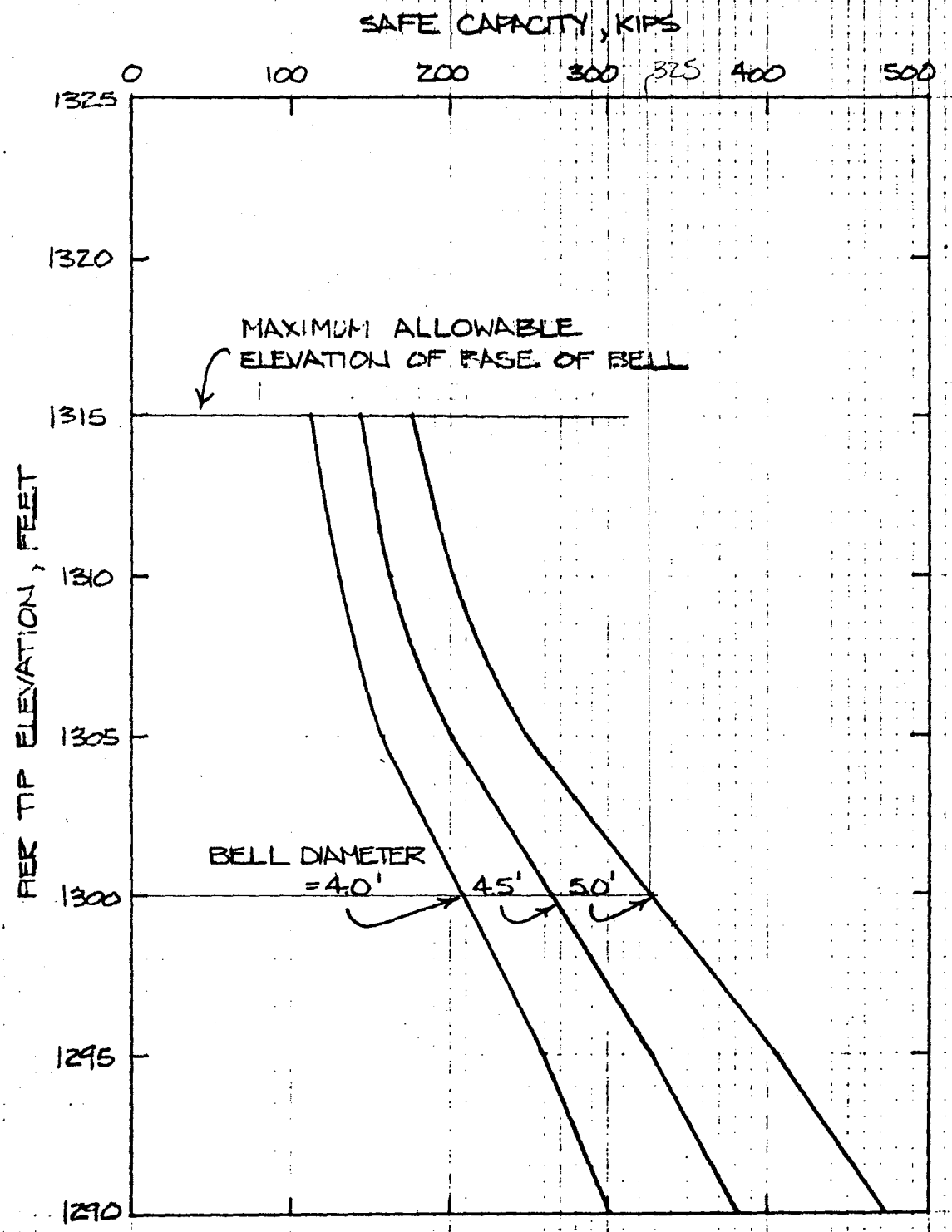
FIGURE 4

SAFE DOWNWARD CAPACITIES FOR BELLED PIERS.



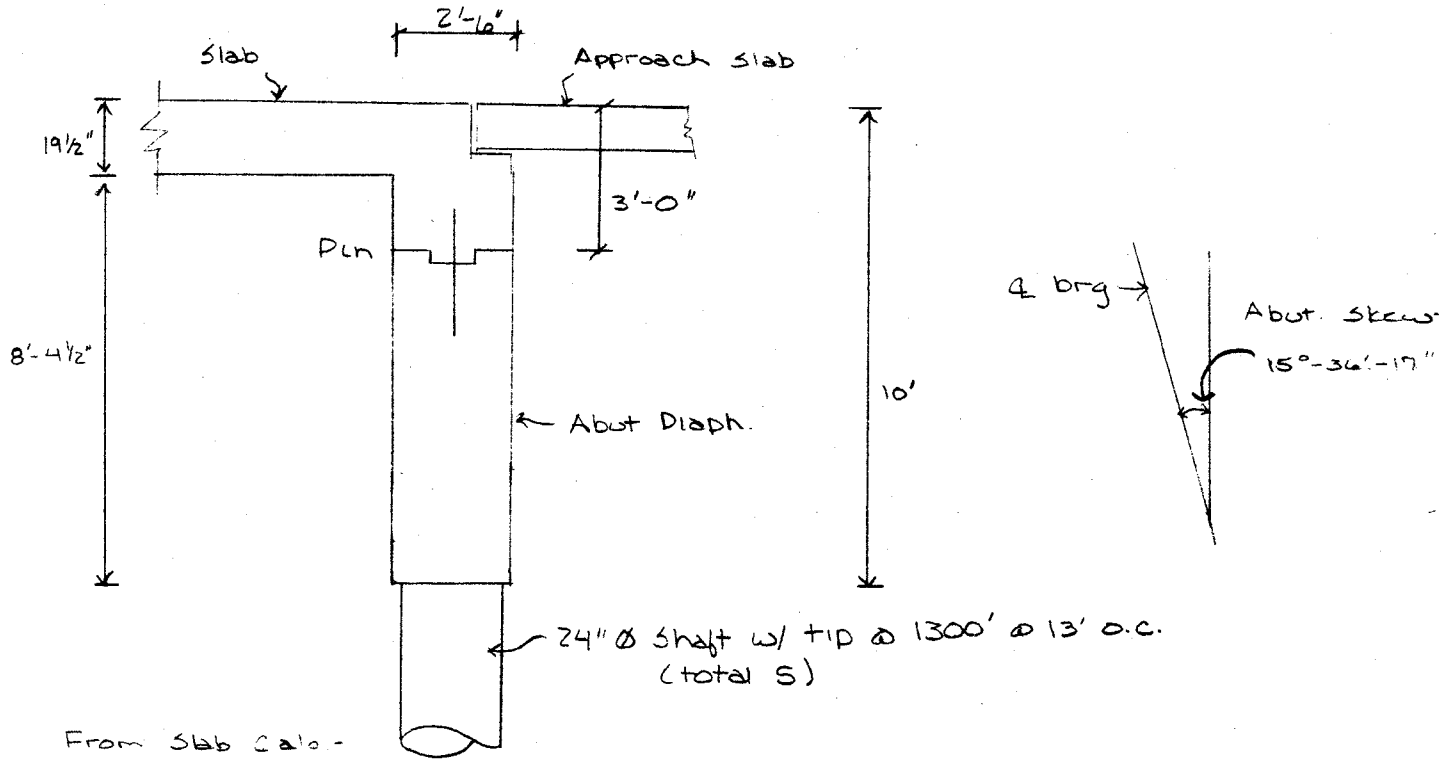
SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

Project GUADALUPE & POWER RD. BRIDGES
Job No: E84-2
Computed by: JRF Ckd. by:
Date 2/13/84 Page of

Abutment Design

Abutment Cap



From Slab Calc -

$D_L = 3.97\ ^k/_{ft}$ Due to Skew $D_L = 0.963(3.99\ ^k/_{ft}) = 3.84\ ^k/_{ft}$

weight of abutment $= (0.150\ ^k/_{ft^3})(10')(2.5') = 3.75\ ^k/_{ft}$

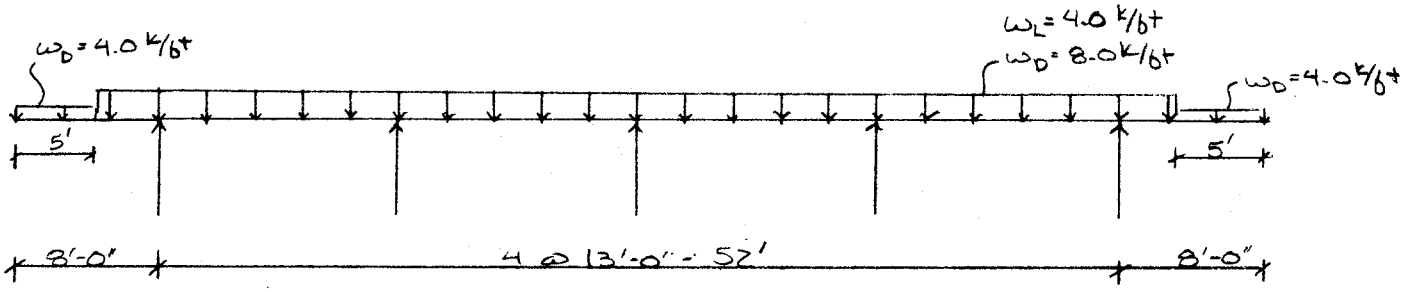
Barrier : $0.4\ ^k/_{ft} \times 2 = 0.8\ ^k/_{ft}$ Load from barrier $= \frac{0.8}{0.27}(3.97)/(55.17/\cos 15.60)$
$= 0.21\ ^k/_{ft}$

$w_D = 3.84 + 3.75 + 0.21 = 7.80\ ^k/_{ft}$ use $w_D = 8.0\ ^k/_{ft}$

Using Table 3.1 - 'Moments, Shears & Reactions for Continuous Highway Bridges'
N = 1.1 Interpolating for 'L' = 37.5'

Max Reaction @ A = 50.8 k/lane (4 lanes)

$w_L = \frac{50.8(4)}{55.17/\cos 15.6^\circ} = 3.55\ ^k/_{ft}$ use $w_L = 4.0\ ^k/_{ft}$

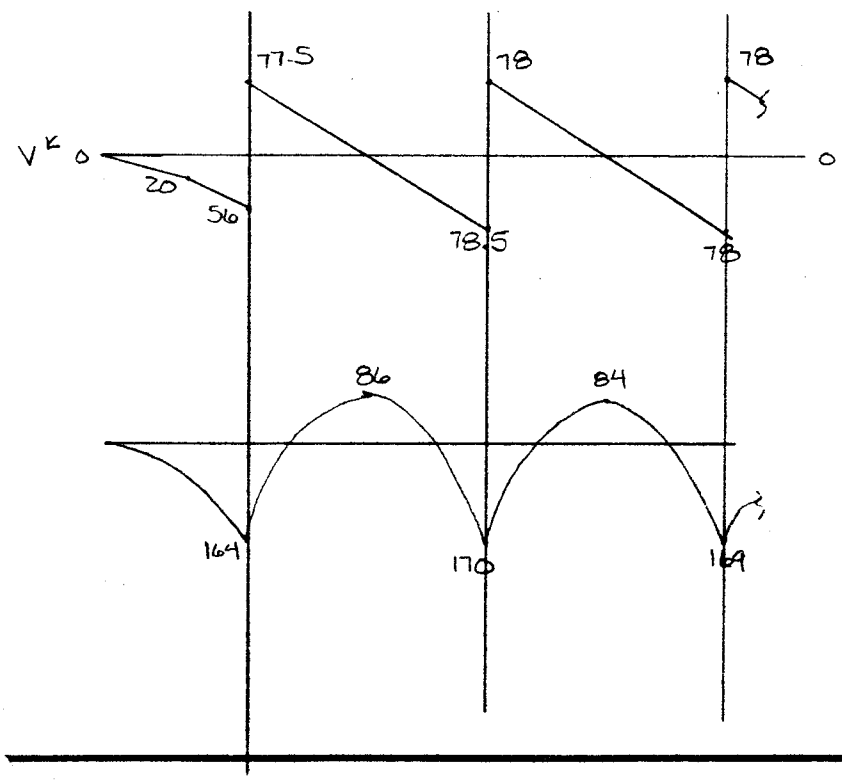


(D+L)

164	-169	169		-169	169	-164
	5	2		-2	-5	
	-164	171		-171	164	

		3/7	4/7	1/2	1/2	4/7	3/4
-164	171	-169	169	-169	169	-171	164
	-1	-1	0	0	1	1	
-164	170	-170	169	-169	170	-170	164

$P_D = 95.5^k$	$P_D = 104.5^k$	$P_D = 104^k$	$P_D = 104.5^k$	$P_D = 95.5$
$P_L = 38^k$	$P_L = 52^k$	$P_L = 52^k$	$P_L = 52^k$	$P_L = 38^k$



Max $+ M_{D+L} = 86$ k-ft

Max $- M_{D+L} = 170$ k-ft

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date 10/5/84 sh. 36 of

Design for $-M = 170^{k\text{-}ft}$

Try 3 #7 $A_s = 3(0.60) = 1.80 \text{ in}^2$ $d = 7'(12''/1) - 6'' - .625 - .875/2 = 76.9''$

Use $d = 76''$

$n = \frac{29000}{57\sqrt{3000}} = 9$ $n\frac{A_s}{bd} = 9\frac{1.80}{(30)(76)} = 0.0071$ $k = 0.112$ $j = 0.962$

$f_s = \frac{170(12)}{1.80(.962)(76.9'')} = 15.3 \text{ ksi} < 20 \text{ ksi}$

$f_c = \frac{0.112(15.3)}{9(1-.112)} = 0.214 \text{ ksi} < 1.2 \text{ ksi}$

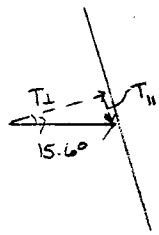
$\therefore$ O.K Use 3 #7

Lateral Load Reinf.:

Lateral Loads - See Caisson Design Calcs

Earth Pressure: $.672^{k}/LF + .966^{k}/LF = 1.64^{k}/LF$

Temperature: 15.6^{k}/caisson



$T_\perp = 15.6^{k} \cos 15.6° = 15.0^{k}$

$w_\perp = 15.0^{k}/13' = 1.2^{k}/LF$

Lateral Load/LF $= 1.64^{k}/LF + 1.2^{k}/LF = 2.84^{k}/LF$

Load Case IV $w = 2.84/1.25 = 2.3^{k}/LF$

Use $w = 3.0^{k}/LF$

Ref: 'Bridge Planning & Design Manual, Vol IV,' pg 6-22

Use #5 @ 12" $M/ft \approx \frac{3.0(7)}{4} = 5.25$ k-ft $A_s/ft = 0.31$ in²

$d = 30'' - 3'' - .625/2 = 26.7''$ Use $d = 26''$

$n\frac{A_s}{bd} = 9\frac{0.31}{(12)(26)} = 0.00894$ $k = 0.125$ $j = 0.958$

$f_s = \frac{5.25(12)}{0.31(.958)(26)} = 8.16$ ksi < 20.0 ksi

∴ O.K.

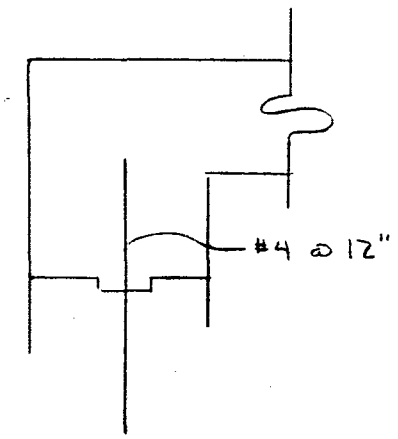
Check Shear:

$V = (0.966 + 1.2)/1.25 = 1.7^k$ $v = 1.7^k/(12)(26) = 0.005$ ksi $< \frac{1}{2}0.95\sqrt{3000} = 26$ psi

∴ No shear Reinf. Req'd:

Note: used $f'_c = 4000$ psi Redesign Not Req'd

Design Pin



AASHTO 1.5.29(D)

$A_{vf} = V/f_s\mu$

$f_s = 20{,}000$ psi $\times 1.25$

$\mu = 1.4$

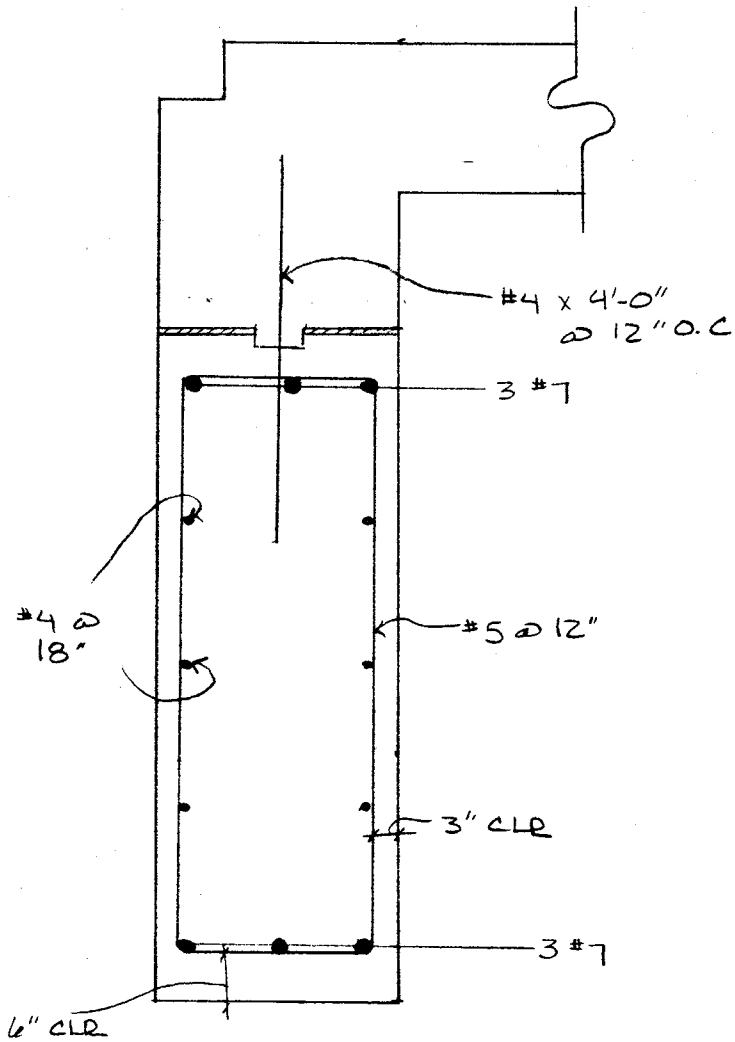
$V = .672^k/LF + 1.2^k/LF = 1.9^k/LF$

Use $V = 2.0^k/LF$

$A_{vf} = 2.0/20(1.25)(1.4) = 0.06$ in²/LF

#4 @ 12" O.K. $A_s = 0.20$ in²/LF

Abutment Section



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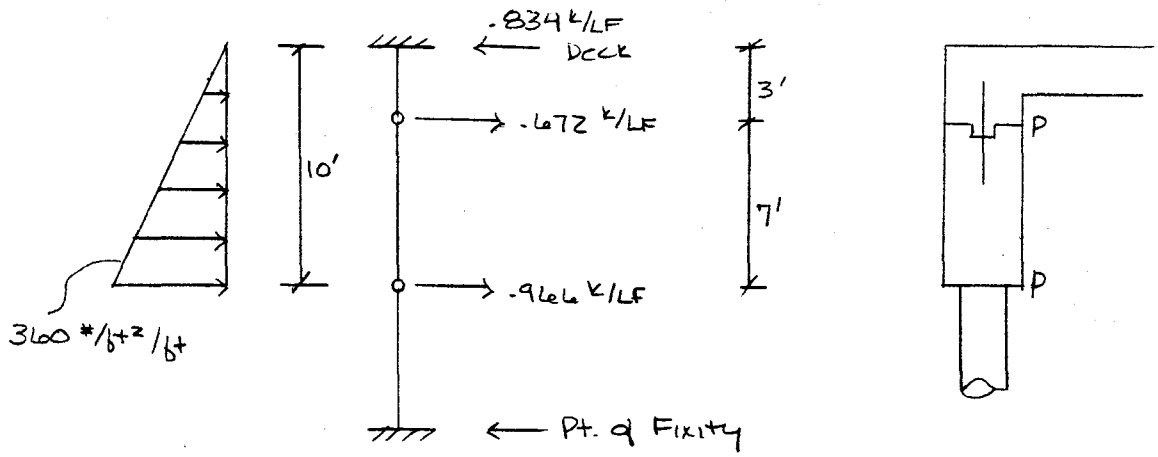
Caisson Design

Assumptions:

- Abutment cap is pinned to caissons - no moment transfer
- No earthquake forces on caissons - total EQ force taken by piers
- Assume future bank protection - No stream flow force

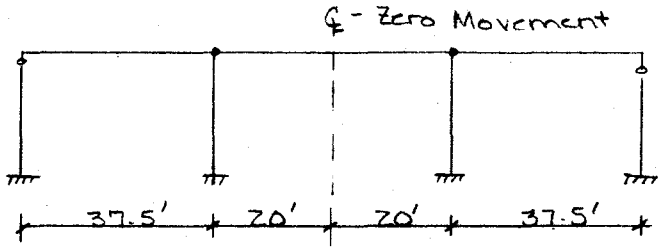
Lateral Load due to Earth Pressure:



Ext. Caissons support 14.5'

Earth Pressure / ext. caisson = (.966 k/LF)(14.5') = 14.0k

Lateral Load due to Temperature



$\Delta = (.000006)(57.5')(12''/1)(40°) = 0.16''$

Use DM-7 to determine P_Δ (See Appendix)

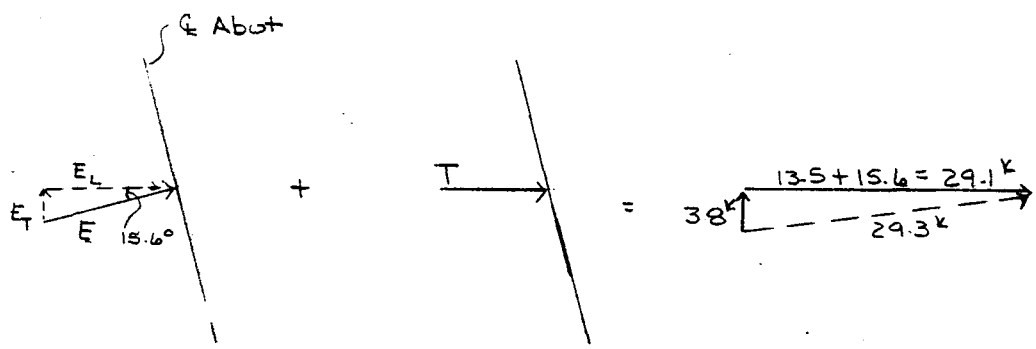
$f = .06\ ^k/in^3 \qquad EI = 3122\ ksi\,(\pi(12'')^4/4) = 5.08 \times 10^7\ ksi$

$T = \left(\frac{5.08 \times 10^7}{.06}\right)^{1/5} = 61'' \qquad L = 30' \qquad L/T = 30(12)/61 = 5.9$

$P_\Delta = \frac{\Delta EI}{F_\Delta T^3} \qquad @\ Z/T = 0 \qquad F_\Delta = 2.3 \qquad P_\Delta = \frac{0.16''(5.08 \times 10^7)}{2.3(61)^3} = 15.6^k$ /column

Load Case IV : $1.0(D + L + E + T) \leq 1.25$

E + T:



$E_L = 14.0^k(\cos 15.6°) = 13.5^k \qquad T = 15.6^k \qquad P = (3.8^2 + 29.1^2)^{1/2} = 29.3^k$

$E_T = 14.0^k(\sin 15.6°) = 3.8^k$

DM-7

$@\ Z/T = 1.3 \qquad M_p = 0.77(29.3^k)(61/12) = 115\ ^k\text{-}ft$

Case IV M = 115/1.25 = 92 k-ft
P = (95.5 + 38)/1.25 = 107 k

- Slenderness Need Not Be Considered - Caisson fully embedded

Design Loads M_u = 92/0.35 = 263 k-ft
P_u = 107/0.35 = 307 k

f_y = 60,000
f_c = 3,000
D = 24" D1 = 4.25
Use 8 #11 AS1 = 1.56 N = 8

Use 1/2" ø Spiral Ties @ 6"

Note: Used f'c = 4000 psi
Redesign not necessary

```
CIRCON

                    "P"
FY=60,000.00
Fc=3,000.00
D=24.00
D1=4.25
AS1=1.56
N=8.00
P=307.00
MX=263.00
MY=1.00E-8

REBAR COORDINATES
Rb=7.75 ∡=0.00
Rb=7.75 ∡=45.00
Rb=7.75 ∡=90.00
Rb=7.75 ∡=135.00
Rb=7.75 ∡=180.00
Rb=7.75 ∡=225.00
Rb=7.75 ∡=270.00
Rb=7.75 ∡=315.00

(.8)PMAX=1,047.52

ebR=13.46
MbR=3,866.30
PbR=287.16
cbR=11.69
B∡=2.18E-9

P=307.47
MX=320.64
MY=1.22E-8
cR=11.89
ecR=1.04
```

> 263 lk
O.K.

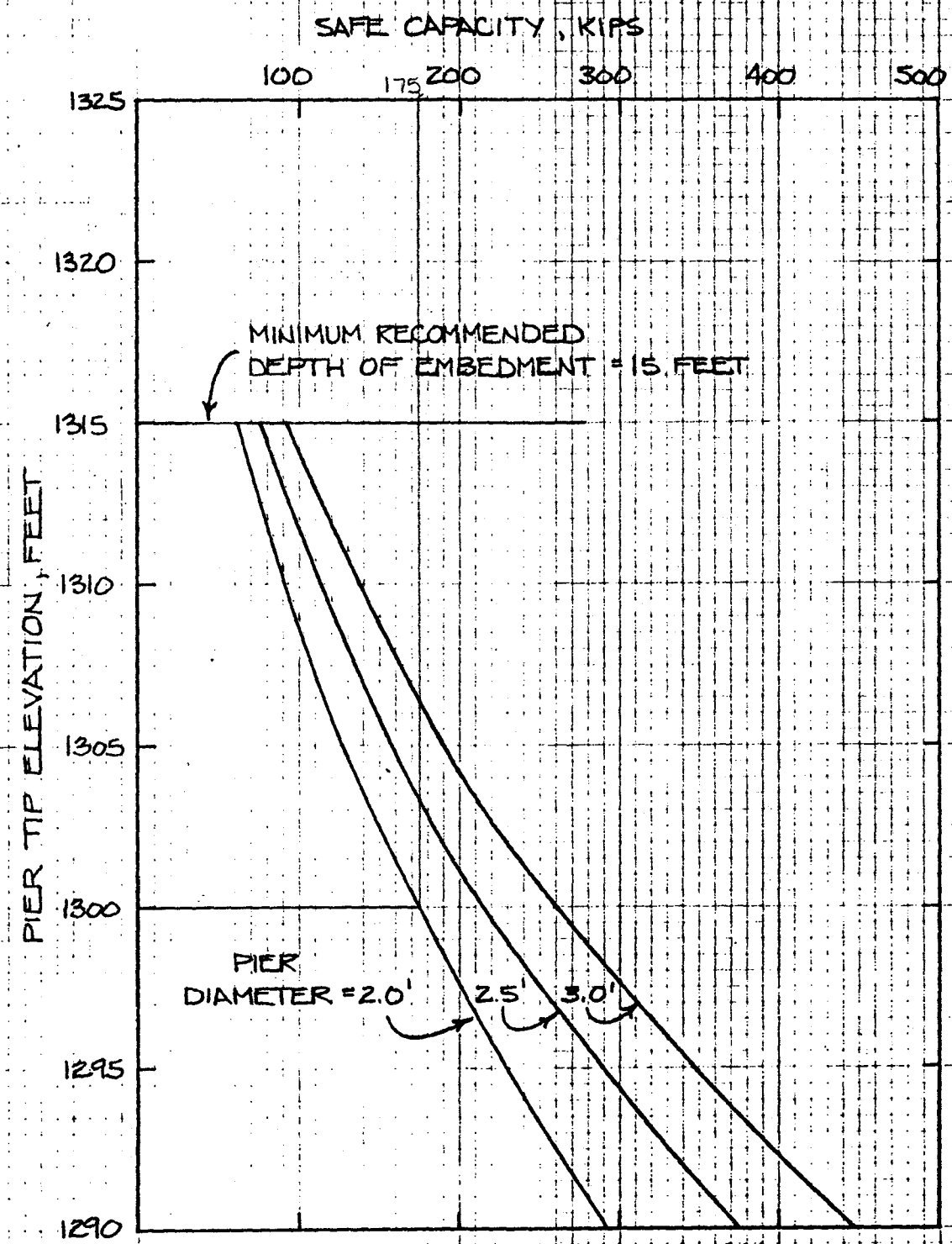
Axial Load = $P_D + P_L = 104.5^k + 52^k = 157^k$

Per SHB Soil Report, Figure 1 (attached)
@ 1300' Safe Bearing Capacity = $175^k > 157^k$

Use 24" Ø Drilled Shafts @ 13'-0" o.c.
w/ 8 #11 Verts & 1/2" Ø Spiral Ties @ 6" pitch
Caisson Tip Elev. = 1300'

FIGURE 1

SAFE DOWNWARD CAPACITIES
FOR STRAIGHT PIERS
- ABUTMENT PIERS -



SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

Project GUADALUPE + POWER RD. BRIDGES
Job No: E84-2
Computed by: RF Ckd. by:
Date 2/13/84 Page of

QUANTITIES

QUANTITIES

CONCRETE

SUPER-STRUCTURE $F'_c = 4500$

DECK $116.167(1.625)(55.167)(1/27) = 385.70$ cy

HAUNCH GIRDER $\{5.25(7.5''/12)(55.167'/\cos 15.605°)$
$-2(1/2)(7.5''/12)(7.5''/12)(55.167/\cos 15.605)$
$+(10''/12)(10''/12)(3''/12)(4)\}(1/27)(2) = 12.3$ cy

ABUTMENTS :

#1 DECK TOE $\{2.5'(1.21')(55.167'/\cos 15.605)$
$+(8''/12)(7.5''/12)(55.167/\cos 15.605)$
$+(2.5''/12)(5.5''/12)(55.167/\cos 15.605)\}(1/27) = 7.5$ cy

$F'_c = 4000$ SUBSTRUCTURE $\{(7')(2.5')(68') +$
$+(68' - 55.167/\cos 15.605)(2.5')(2.47')\}(1/27) = 46.5$ cy

#2 SIMILAR TO #1 DECK TOE 7.5 cy
SUBSTRUCTURE 46.5 cy

COLUMNS: $3.14(2.5')(9''/12)(4)(2)(1/27) = 1.8$ cy.

SUMMARY

	$f'_c = 4500$	$f'_c = 4000$
ABUTMENT #1	7.5 cy	46.5 cy
" #2	7.5 cy	46.5 cy
PIER #1	—	1.0 cy
" #2	—	1.0 cy
DECK & HAUNCH	398 cy	—

APPROACH SLABS: $15'[(55.167 - 3.167)](1')(2)(1/27) = 57.8$ cy

BARRIERS: $2(117.5)(2.5\ ft^2)(1/27) = 21.8$ cy

DRILLED SHAFTS

30" ϕ W/5'-0" BELL $4 \times 2 \times 35' = 280'$
24" ϕ SHAFT $5 \times 2 \times 27.74' = 277.4'$

24" CONCRETE: $(28)(3.14)(1)^2/27 = 3.26$ cy/SHAFT
STEEL $(28)(8)(2.044) + 57(\pi)(2)(0.667) = 696.7$ #/SHAFT

30" CONCRETE: $\{(35)(1.25)^2(3.14) + (3.14)(.5)(2.5^2 - 1.25^2) + (3.14)(1.25)(2.5^2 - 1.25^2)\}1/27 = 6.98$ cy/SHAFT
STEEL: $\{35(12)(5.313) + 2(3.14)(70)(0.668)\} = 2524.7$ #/SHAFT

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project GUADALUPE 83-58
designed by WFS
checked by
date 10-4-84 sh. 4a of

QUANTITIES CONT'D

REINFORCING STEEL

DECK:

TOP #10s: 2 (4)(59.75')(4.303)
+ 1 (2)(59.75')(4.303)
+ 35 (2)(19.5') (4.303)
+ 35 (12.5') (2) (4.303)
+ 35 (2)(59.75')(4.303) = 30207.1#

BOTT. #10s: 2 (4) (39.583)(4.303)
+ 2 (1) (39.583)(4.303)
+ 2 (35)(19.0') (4.303)
+ 2 (35)(30.0') (4.303)
+ 2 (35)(39.583)(4.303) = 28385.3#

4 (43.167')(4.303) = 743.0#

#8s: 1 (43.167)(2.67)
+ 35 (22.0')(2.67)
+ 35 (32.0') (2.67)
+ 35 (42.0) (2.67) = 9086.5#

TRANS (#4@18) (27 + 2(25))(55.167')(.668) = 2839#
(#5s) (31 + 2(32+10))(55.167) (1.043) = 6621#
77881#

HAUNCH GIRDERS: #10 LONG 2 [10 (55.167/cos 15.605)(4.303) = 4931#
#8 LONG 2[10 (55.167/cos 15.605)] (2.67) = 3060#
#5 [] 2 [103 (4.5')](1.043) = 967#
#5 [ ] 2 (103)(7.33') (1.043) = 1575#
10533.0#

DECK TOE INS: #5 LONG 2 (1)(57.3') (1.043) = 60#
" " 2 (4)(68')(1.043) = 567#
#4 ⊏ 2 (56)(8')(0.668) = 599#
1226#

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QUANTITIES CONT'D

ABUTMENTS	#5 LONG	2(10)(68')(1.043)	=	1418#
	#5 [ ]	2(56)(15.5')(1.043)	=	1811#
	#7 LONG	2(8)(67.5')(3.40)	=	3672#
	#4 DOWEL	2(56)(4')(0.668)	=	299#
	#5 U	2(12)(20')(1.043)	=	501#
	#5 TOP	2(68)(2.5)(1.043)	=	355#
	#5 ENDS	2(5)(2)(2.5)(1.043)	=	52#
	#5	2(4)(14')(1.043)	=	117#
				8225#

PIERS	#11 VERTS	8(12)(9"/12)(5.313)	=	383#
	#4 TIES	8(1)(8')(0.668)	=	43#
	#8 PINREINF X	2(6.2')(4)(6)(2.67)	=	790#
				1216#

SUMMARY:

ABUT #1 = 8225/2 = 4112.5#
ABUT #2 = 4112.5#
PIER #1 = 1216/2 = 608#
PIER #2 = 608#
SUPERSTRUCT = 77881 + 10533 + 1226 = 89640#

APPROACH SLAB:

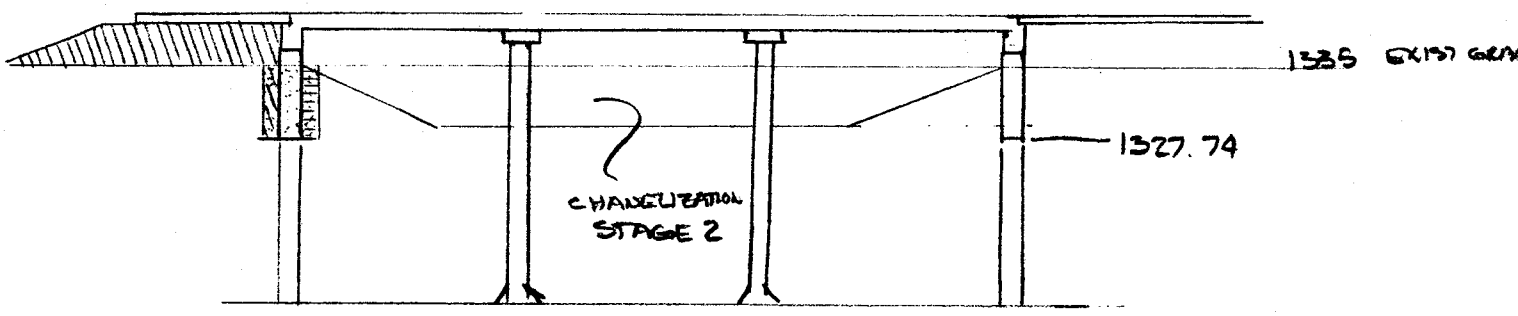
#4 @ 18	2(5.5')(36)(0.668)	=	265#
#4 @ 18 LONG	2(14.5)(36)(0.668)	=	697#
#8 @ 9"	2(15)(71)(2.67)	=	5687#
#5 @ 12" TRANS.	2(52)(16)(1.043)	=	1736#
#4 @ 18 TRANS	2(52)(11)(0.668)	=	764#
#5 ANCHOR	2(2.5')(53)(1.043)	=	276#
#4 DOWEL	2(2.5)(53)(0.668)	=	177#
			9602#

QUANTITIES CONT'D

BARRIERS:

# 4 LONG	2 [6(115) + 2(1.5)](.668) =	9[illegible]6#
# 5 LONG	2 [2(115 + 2(2')](1.043) =	496#
# 6 LONG	2 [115 + 2 (2.5')](1.502) =	360#
# 10 LONG	2 [2(115) + 2 (4.5')](4.303) =	2134#
# 4 @ 9" /	2 [154 (3.5)](.668) =	720#
# 4 @ 18" |	2 (77)(2.5')(.668) =	257#
# 4 @ 9" ⊔	2 (154)(6.5')(.668) =	1337#
# 4 @ 12" [	2 (116)(5')(.668) =	775#
		7025#

EXCAVATION:



STAGE 1

EXCAVATION: $\{(1335 - 1327.74)(68' + 2(1.5'))(2.5' + 2(1.5'))\}(2)(1/27)$
$= 210.0$ cy

SPECIAL BACKFILL: $\{(68 + 2(1.5))(1335 - 1327.74)(1.5)$
$+ 2.1(17')(57) + 1/2(2)(4)(57)\}(2)(1/27) = 225$ cy

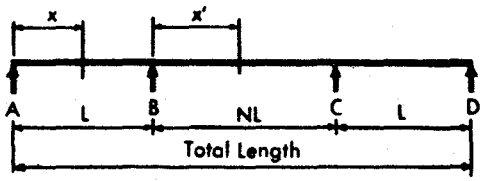
STRUCTURE BACKFILL: $2(68 + 2(1.5))(1335 - 1327.74)(1.5)(1/27) = 57$ cy

Appendix

TABLE 3.1

Symmetrical three-span continuous beam.
Constant moment of inertia.
AASHO HS20-44 loading.

N=1.1



Total Length Ft.	Exter. Span Length "L"	Max. Reaction Kips.		Max. Shear Kips.		Max. Moment Kip.-ft.			Impact Coefficient				Dist.-Ft.	
		at A	at B	in AB at B	in BC at B or C	in AB at X	at B	in BC at X'	I	II	III	IV	X	X'
90	29.0	45.7	62.2	-52.0	51.1	219.1	-185.2	195.6	.300	.300	.300	.269	10.3	14.5
105	33.9	48.8	64.6	-55.7	54.4	275.9	-219.4	250.4	.300	.300	.300	.255	12.2	17.1
120	38.7	51.4	66.2	-58.3	56.9	338.2	-254.0	308.0	.300	.300	.298	.242	15.6	19.8
135	43.5	53.4	67.3	-60.2	58.8	406.2	-302.8	367.4	.297	.293	.289	.231	17.5	22.5
150	48.4	55.2	68.2	-61.7	60.4	475.0	-355.4	428.0	.288	.284	.281	.221	19.5	25.1
165	53.2	56.6	68.8	-62.9	61.6	544.3	-411.8	489.7	.281	.276	.272	.211	21.6	27.8
180	58.1	57.8	72.5	-63.8	62.7	614.2	-472.1	552.0	.273	.269	.265	.202	23.6	30.4
195	62.9	58.9	76.3	-64.6	63.5	684.3	-536.1	614.9	.266	.262	.257	.194	25.6	33.1
210	67.7	59.8	80.2	-65.3	64.2	754.7	-603.9	678.3	.259	.255	.251	.187	27.7	35.8
225	72.6	60.6	84.1	-65.8	64.8	825.3	-675.5	741.9	.253	.248	.244	.180	29.7	38 4
240	77.4	61.2	87.9	-66.3	65.4	896.1	-750.9	805.9	.247	.242	.238	.174	31.8	41.1
255	82.3	61.9	91.8	-66.7	65.8	967.0	-830.1	870.1	.241	.237	.232	.168	33.8	43.7
270	87.1	62.4	95.7	-67.1	66.2	1038.1	-913.2	934.5	.236	.231	.226	.162	35.9	46.4
285	91.9	62.9	99.5	-67.4	66.6	1109.2	-999.9	999.1	.230	.226	.221	.157	38.0	49.1
300	96.8	63.4	103.4	-67.7	66.9	1180.5	-1090.5	1063.8	.225	.221	.216	.152	40.0	51.7
315	101.6	63.8	107.3	-67.9	67.2	1251.8	-1184.9	1128.6	.221	.216	.211	.148	42.1	54.4
330	106.5	64.1	111.2	-68.7	68.5	1323.1	-1283.1	1193.6	.216	.211	.207	.143	44.2	57.1
345	111.3	64.5	115.0	-70.6	70.4	1394.5	-1385.1	1258.6	.212	.207	.202	.139	46.2	59.7
360	116.1	64.8	118.9	-72.6	72.3	1466.0	-1490.9	1323.7	.207	.202	.198	.136	48.3	62.4
Impact		I	IV	I	III	I	II	III						
Dead Load		.3900 x wL	1.1600 x wL	-.6100 x wL	.5500 x wL	.0761 x wL^2	-.1100 x wL^2	.0413 x wL^2					.3900 L	.5500 x L

From 'Moments, Shears, & Reactions for Continuous Highway Bridges', AISC 1966

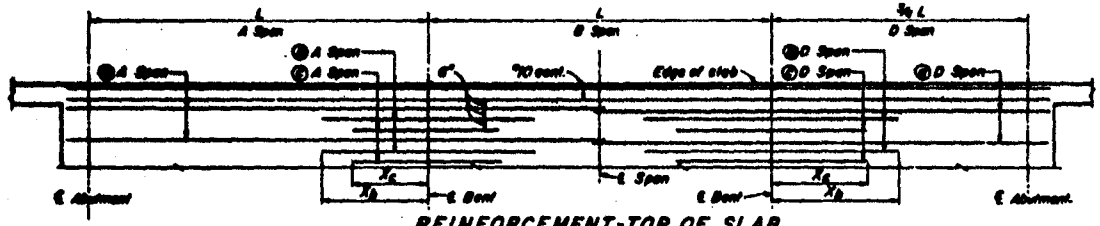
Guadalupe Rd. Br. 83-58
RR

Length of Span		16'			18'			20'			22'			24'			26'			28'			30'			32'			34'			36'			38'			40'			42'			44'		
REINFORCEMENT		A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D	A	B	D
Top of Slab	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	17'-0"	—	8'-0"	20'-0"	—	8'-0"	20'-0"	—	17'-0"	20'-0"	—	17'-0"	20'-0"	—	20'-0"	20'-0"	—	17'-0"	20'-0"	—	17'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"	20'-0"	—	20'-0"
Bottom of Slab	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	12'-0"	—	8'-0"	14'-0"	—	8'-0"	15'-0"	—	12'-0"	15'-0"	—	12'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"	15'-0"	—	14'-0"
Top of Slab	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	6'-0"	—	6'-0"	6'-0"	—	6'-0"	6'-6"	—	6'-0"	7'-6"	—	6'-0"	7'-0"	—	6'-6"	8'-6"	—	7'-0"	6'-6"	—	7'-0"	5'-6"	—	7'-0"	6'-6"	—	7'-0"	7'-6"	—	7'-0"	8'-6"	—	7'-0"	8'-6"	—	7'-0"	9'-6"	—	8'-0"	9'-6"	—	8'-0"	10'-6"	—	8'-0"
Bottom of Slab	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	6'-0"	—	6'-0"	7'-0"	—	6'-0"	7'-0"	—	6'-0"	8'-0"	—	7'-0"	8'-0"	—	7'-0"	8'-0"	—	7'-0"	8'-0"	—	7'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	7'-0"	10'-0"	—	8'-0"	11'-0"	—	9'-0"	11'-0"	—	9'-0"	11'-0"	—	9'-0"	11'-0"	—	9'-0"
	Length	3'-6"	—	3'-6"	3'-6"	—	3'-6"	4'-0"	—	3'-6"	4'-0"	—	3'-6"	4'-0"	—	3'-6"	4'-0"	—	3'-6"	4'-0"	—	3'-6"	4'-0"	—	3'-6"	4'-6"	—	4'-0"	5'-6"	—	4'-0"	5'-6"	—	4'-0"	6'-0"	—	4'-0"	6'-0"	—	4'-0"	6'-0"	—	4'-0"	6'-0"	—	4'-0"
Bottom of Slab	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	10'-0"	10'-0"	10'-0"	11'-0"	11'-0"	11'-0"	11'-0"	11'-0"	11'-0"	12'-0"	12'-0"	12'-0"	13'-0"	13'-0"	13'-0"	13'-0"	14'-0"	13'-0"	13'-0"	14'-0"	13'-0"	13'-0"	14'-0"	13'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"	14'-0"
	Length	11'-0"	8'-0"	11'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	11'-0"	12'-0"	11'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"	12'-0"	12'-0"	10'-0"
	Bars	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#7	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7	#8	—	#7
	Length	7'-0"	—	7'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"	8'-0"	—	8'-0"
	Length	6'-0"	6'-0"	6'-0"	7'-6"	7'-6"	7'-6"	7'-6"	7'-6"	7'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"	8'-6"
T = Thickness of slab		10½"			11½"			12"			12½"			13½"			14"			15"			16"			16½"			17"			17½"			18"			19"			19½"			20"		
Approx. Quantities per ft of width	Conc. ft³	14.3	14.1	11.6	16.3	17.3	13.1	18.0	20.0	15.0	19.8	23.0	16.8	20.1	27.0	21.4	21.5	30.3	23.9	24.2	33.0	27.5	27.3	36.0	31.4	29.4	44.0	33.8	32.3	40.7	39.3	34.0	53.3	40.8	36.8	59.0	44.3	39.9	66.3	48.8	43.8	68.7	53.8	48.0	73.3	58.7
	Steel lbs	139	118	88	158	139	106	180	160	118	200	180	128	200	208	140	190	245	165	310	255	175	340	265	190	370	315	210	400	340	220	430	375	230	490	415	260	550	455	290	590	495	300	635	535	340
Deflection at Midspan Δ	Bare Deck							.02'	.00'	—	.03'	.00'	—	.03'	.00'	—	.04'	.00'	—	.05'	.00'	—	.06'	.00'	—	.07'	.00'	—	.09'	.00'	—	.10'	.00'	—	.11'	.01'	—	.13'	.01'	—	.15'	.01'	—	.16'	.01'	—
	Wearing Surface							Negligible Deflection						.02'	—	—	—	.02'	—	—	.02'	—	—	.03'	.01'	—	.03'	.01'	—	.04'	.02'	—	.05'	.02'	—	.06'	.02'	—	.06'	.03'	—	.07'	.03'	—	.08'	.03'

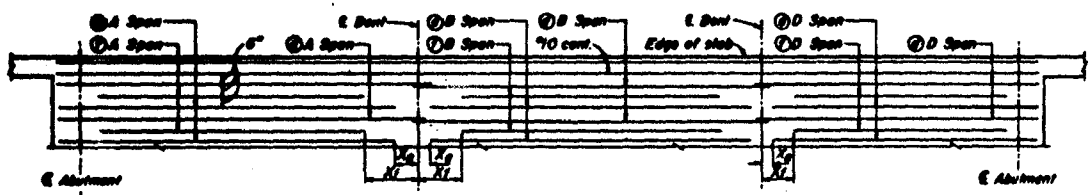
● Add ½" for "Marine Environment" & adjust concrete quantity.

Note:
"D" Span lengths are actually ⅔ of the length shown in the heading. (Heading 24': "A" & "B" Spans are actually 24', "D" Span is 16'.)

* Length includes allowance for one additional splice in "A" or "D" Span.



REINFORCEMENT-TOP OF SLAB



REINFORCEMENT-BOTTOM OF SLAB

LIVE LOADING: HS20-44 & ALT.

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
DIVISION OF ENGINEERING SERVICES

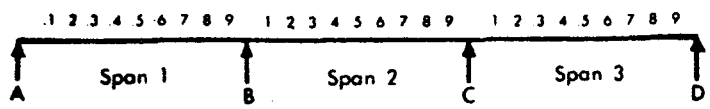
STANDARD SLAB BRIDGE

SLAB DETAILS-3 SPANS

Guadalupe RR
RR 83-58

Unit load at		MOMENTS/PL																				
		SPAN 1											SPAN 2									
		A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
SPAN 1	A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	.1	0	.0874	.0747	.0621	.0494	.0368	.0242	.0115	-.0011	-.0138	-.0264	-.0231	-.0198	-.0165	-.0132	-.0099	-.0066	-.0033	.0000	.0033	.0066
	.2	0	.0749	.1498	.1246	.0995	.0744	.0493	.0242	-.0010	-.0261	-.0512	-.0448	-.0384	-.0320	-.0256	-.0192	-.0128	-.0064	.0000	.0064	.0128
	.3	0	.0627	.1254	.1882	.1509	.1136	.0763	.0390	.0018	-.0355	-.0728	-.0637	-.0546	-.0455	-.0364	-.0273	-.0182	-.0091	.0000	.0091	.0182
	.4	0	.0510	.1021	.1531	.2042	.1552	.1062	.0573	.0083	-.0406	-.0896	-.0784	-.0672	-.0560	-.0448	-.0336	-.0224	-.0112	.0000	.0112	.0224
	.5	0	.0400	.0800	.1200	.1600	.2000	.1400	.0800	.0200	-.0400	-.1000	-.0875	-.0750	-.0625	-.0500	-.0375	-.0250	-.0125	.0000	.0125	.0250
	.6	0	.0298	.0595	.0893	.1190	.1488	.1786	.1084	.0381	-.0322	-.1024	-.0896	-.0768	-.0640	-.0512	-.0384	-.0256	-.0128	.0000	.0128	.0256
	.7	0	.0205	.0410	.0614	.0819	.1024	.1229	.1434	.0638	-.0157	-.0952	-.0833	-.0714	-.0595	-.0476	-.0357	-.0238	-.0119	.0000	.0119	.0238
	.8	0	.0123	.0246	.0370	.0493	.0616	.0739	.0862	.0986	.0109	-.0768	-.0672	-.0576	-.0480	-.0384	-.0288	-.0192	-.0096	.0000	.0096	.0192
	.9	0	.0054	.0108	.0163	.0218	.0272	.0326	.0381	.0435	.0490	-.0456	-.0399	-.0342	-.0285	-.0228	-.0171	-.0114	-.0057	.0000	.0057	.0114
	B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SPAN 2	.1	0	-.0039	-.0078	-.0117	-.0156	-.0195	-.0234	-.0273	-.0312	-.0351	-.0390	.0534	.0458	.0382	.0306	.0230	.0154	.0073	.0002	-.0074	-.0150
	.2	0	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512	-.0576	-.0640	.0192	.1024	.0856	.0688	.0520	.0352	.0184	.0016	-.0152	-.0320
	.3	0	-.0077	-.0154	-.0231	-.0308	-.0385	-.0462	-.0539	-.0616	-.0691	-.0770	-.0042	.0686	.1414	.1142	.0870	.0598	.0326	.0054	-.0218	-.0490
	.4	0	-.0080	-.0160	-.0240	-.0320	-.0400	-.0480	-.0560	-.0640	-.0720	-.0800	-.0184	.0432	.1048	.1664	.1280	.0896	.0512	.0128	-.0256	-.0640
	.5	0	-.0075	-.0150	-.0225	-.0300	-.0375	-.0450	-.0525	-.0600	-.0675	-.0756	-.0250	.0250	.0750	.1250	.1750	.1250	.0750	.0250	-.0250	-.0750
	.6	0	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512	-.0576	-.0640	-.0256	.0128	.0512	.0896	.1280	.1664	.1048	.0432	-.0184	-.0800
	.7	0	-.0049	-.0098	-.0147	-.0196	-.0245	-.0294	-.0343	-.0392	-.0441	-.0490	-.0218	.0054	.0326	.0598	.0870	.1142	.1414	.0686	-.0042	-.0770
	.8	0	-.0032	-.0064	-.0096	-.0128	-.0160	-.0192	-.0224	-.0256	-.0288	-.0320	-.0152	.0016	.0184	.0352	.0520	.0688	.0856	.1024	.0192	-.0640
	.9	0	-.0015	-.0030	-.0045	-.0060	-.0075	-.0090	-.0105	-.0120	-.0135	-.0150	-.0074	.0002	.0078	.0154	.0230	.0306	.0382	.0458	.0534	-.0390
	C	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SPAN 3	.1	0	.0011	.0023	.0034	.0046	.0057	.0068	.0080	.0091	.0103	.0114	.0057	.0000	-.0057	-.0114	-.0171	-.0228	-.0285	-.0342	-.0399	-.0456
	.2	0	.0019	.0038	.0058	.0077	.0096	.0115	.0134	.0154	.0173	.0192	.0096	.0000	-.0096	-.0192	-.0288	-.0384	-.0480	-.0576	-.0672	-.0768
	.3	0	.0024	.0048	.0071	.0095	.0119	.0143	.0167	.0190	.0214	.0238	.0119	.0000	-.0119	-.0238	-.0357	-.0476	-.0595	-.0714	-.0833	-.0952
	.4	0	.0026	.0051	.0077	.0102	.0128	.0154	.0179	.0205	.0230	.0256	.0128	.0000	-.0128	-.0256	-.0384	-.0512	-.0640	-.0768	-.0896	-.1024
	.5	0	.0025	.0050	.0075	.0100	.0125	.0150	.0175	.0200	.0225	.0250	.0125	.0000	-.0125	-.0250	-.0375	-.0500	-.0625	-.0750	-.0875	-.1000
	.6	0	.0022	.0045	.0067	.0090	.0112	.0134	.0157	.0179	.0202	.0224	.0112	.0000	-.0112	-.0224	-.0336	-.0448	-.0560	-.0672	-.0784	-.0896
	.7	0	.0018	.0036	.0055	.0073	.0091	.0109	.0127	.0146	.0164	.0182	.0091	.0000	-.0091	-.0182	-.0273	-.0364	-.0455	-.0546	-.0637	-.0728
	.8	0	.0013	.0026	.0038	.0051	.0064	.0077	.0090	.0102	.0115	.0128	.0064	.0000	-.0064	-.0128	-.0192	-.0256	-.0320	-.0384	-.0448	-.0512
	.9	0	.0007	.0013	.0020	.0026	.0033	.0040	.0046	.0053	.0059	.0066	.0033	.0000	-.0033	-.0066	-.0099	-.0132	-.0165	-.0198	-.0231	-.0264
	D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
+ Area		0	.0400	.0700	.0900	.1000	.1000	.0900	.0700	.0402	.0204	.0167	.0152	.0300	.0550	.0700	.0750	.0700	.0550	.0300	.0152	.0167
− Area		0	-.0050	-.0100	-.0150	-.0200	-.0250	-.0300	-.0350	-.0402	-.0654	-.1167	-.0702	-.0500	-.0500	-.0500	-.0500	-.0500	-.0500	-.0500	-.0702	-.1167
Total Area		0	.0350	.0600	.0750	.0800	.0750	.0600	.0350	.0000	-.0450	-.1000	-.0550	-.0200	.0050	.0200	.0250	.0200	.0050	-.0200	-.0550	-.1000

TABLE A3.0

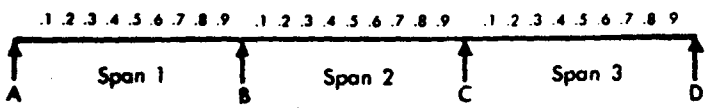


From 'Moments, Shears & Reactions for Continuous Highway Bridges', AISC 1966

Guadalupe Rd. Bridge 83-58
RR

Unit load at	MOMENTS/PL																				
	SPAN 1											SPAN 2									
	A	.1	.2	.3	.4	.5	.6	.7	.8	.9	B	.1	.2	.3	.4	.5	.6	.7	.8	.9	C
A	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	.0675	.0749	.0624	.0499	.0373	.0248	.0123	-.0002	-.0128	-.0253	-.0221	-.0189	-.0157	-.0125	-.0093	-.0061	-.0030	.0002	.0034	.0066
.2	0	.0751	.1502	.1253	.1004	.0755	.0506	.0256	.0007	-.0242	-.0491	-.0429	-.0367	-.0305	-.0243	-.0181	-.0119	-.0057	.0005	.0067	.0129
.3	0	.0630	.1260	.1891	.1521	.1151	.0781	.0411	.0042	-.0328	-.0698	-.0610	-.0522	-.0434	-.0346	-.0258	-.0169	-.0081	.0007	.0095	.0183
.4	0	.0514	.1028	.1542	.2056	.1571	.1085	.0599	.0113	-.0373	-.0859	-.0751	-.0642	-.0534	-.0425	-.0317	-.0209	-.0100	.0008	.0117	.0225
.5	0	.0404	.0808	.1212	.1617	.2021	.1425	.0829	.0233	-.0363	-.0959	-.0838	-.0717	-.0596	-.0475	-.0354	-.0233	-.0112	.0009	.0130	.0251
.6	0	.0302	.0604	.0906	.1207	.1509	.1811	.1113	.0415	-.0283	-.0982	-.0858	-.0734	-.0610	-.0486	-.0362	-.0238	-.0115	.0009	.0133	.0257
.7	0	.0209	.0417	.0626	.0835	.1044	.1252	.1461	.0670	-.0121	-.0913	-.0797	-.0682	-.0567	-.0452	-.0337	-.0222	-.0106	.0009	.0124	.0239
.8	0	.0126	.0253	.0379	.0506	.0632	.0758	.0885	.1011	.0137	-.0736	-.0643	-.0550	-.0458	-.0365	-.0272	-.0179	-.0086	.0007	.0100	.0193
.9	0	.0056	.0113	.0169	.0225	.0281	.0338	.0394	.0450	.0507	-.0437	-.0382	-.0327	-.0272	-.0216	-.0161	-.0106	-.0051	.0004	.0059	.0114
B	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	-.0045	-.0090	-.0135	-.0179	-.0224	-.0289	-.0314	-.0359	-.0404	-.0449	.0569	.0487	.0406	.0324	.0242	.0160	.0078	-.0004	-.0086	-.0168
.2	0	-.0074	-.0147	-.0221	-.0294	-.0368	-.0441	-.0515	-.0588	-.0662	-.0735	.0182	.1100	.0917	.0735	.0552	.0370	.0187	.0004	-.0178	-.0361
.3	0	-.0088	-.0177	-.0265	-.0353	-.0442	-.0530	-.0618	-.0706	-.0795	-.0883	-.0080	.0722	.1525	.1228	.0931	.0634	.0336	.0039	-.0258	-.0555
.4	0	-.0092	-.0183	-.0275	-.0366	-.0458	-.0549	-.0641	-.0732	-.0824	-.0916	-.0237	.0442	.1121	.1799	.1378	.0957	.0536	.0114	-.0307	-.0728
.5	0	-.0086	-.0171	-.0257	-.0342	-.0428	-.0514	-.0599	-.0685	-.0771	-.0856	-.0306	.0244	.0794	.1344	.1894	.1344	.0794	.0244	-.0306	-.0856
.6	0	-.0073	-.0146	-.0218	-.0291	-.0364	-.0437	-.0510	-.0583	-.0655	-.0728	-.0307	.0114	.0536	.0957	.1378	.1799	.1121	.0442	-.0237	-.0916
.7	0	-.0055	-.0111	-.0167	-.0222	-.0278	-.0333	-.0389	-.0444	-.0500	-.0555	-.0258	.0039	.0336	.0634	.0931	.1228	.1525	.0722	-.0080	-.0883
.8	0	-.0036	-.0072	-.0108	-.0144	-.0180	-.0216	-.0252	-.0288	-.0325	-.0361	-.0178	.0004	.0187	.0370	.0552	.0735	.0917	.1100	.0182	-.0735
.9	0	-.0017	-.0034	-.0050	-.0067	-.0084	-.0101	-.0117	-.0134	-.0151	-.0168	-.0086	-.0004	.0078	.0160	.0242	.0324	.0406	.0487	.0569	-.0449
C	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.1	0	.0011	.0023	.0034	.0046	.0057	.0068	.0080	.0091	.0103	.0114	.0059	.0004	-.0051	-.0106	-.0161	-.0216	-.0272	-.0327	-.0382	-.0437
.2	0	.0019	.0039	.0058	.0077	.0097	.0116	.0135	.0154	.0174	.0193	.0100	.0007	-.0086	-.0179	-.0272	-.0365	-.0458	-.0550	-.0643	-.0736
.3	0	.0024	.0048	.0072	.0096	.0120	.0143	.0167	.0191	.0215	.0239	.0124	.0009	-.0106	-.0222	-.0337	-.0452	-.0567	-.0682	-.0797	-.0913
.4	0	.0026	.0051	.0077	.0103	.0129	.0154	.0180	.0206	.0231	.0257	.0133	.0009	-.0115	-.0238	-.0362	-.0486	-.0610	-.0734	-.0858	-.0982
.5	0	.0025	.0050	.0075	.0100	.0126	.0151	.0176	.0201	.0226	.0251	.0130	.0009	-.0112	-.0233	-.0354	-.0475	-.0596	-.0717	-.0838	-.0959
.6	0	.0023	.0045	.0068	.0090	.0113	.0135	.0158	.0180	.0203	.0225	.0117	.0008	-.0100	-.0209	-.0317	-.0425	-.0534	-.0642	-.0751	-.0859
.7	0	.0018	.0037	.0055	.0073	.0092	.0110	.0128	.0146	.0165	.0183	.0095	.0007	-.0081	-.0169	-.0258	-.0346	-.0434	-.0522	-.0610	-.0698
.8	0	.0013	.0026	.0039	.0052	.0065	.0077	.0090	.0103	.0116	.0129	.0067	.0005	-.0057	-.0119	-.0181	-.0243	-.0305	-.0367	-.0429	-.0491
.9	0	.0007	.0013	.0020	.0026	.0033	.0040	.0046	.0053	.0059	.0066	.0034	.0002	-.0030	-.0061	-.0093	-.0125	-.0157	-.0189	-.0221	-.0253
D	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
+ Area	0	.0403	.0705	.0908	.1011	.1015	.0917	.0720	.0423	.0209	.0167	.0163	.0346	.0643	.0824	.0885	.0824	.0643	.0346	.0163	.0167
− Area	0	-.0063	-.0125	-.0188	-.0251	-.0315	-.0377	-.0440	-.0503	-.0749	-.1267	-.0718	-.0478	-.0472	-.0472	-.0472	-.0472	-.0472	-.0478	-.0718	-.1267
Total Area	0	.0340	.0580	.0720	.0760	.0700	.0540	.0280	-.0080	-.0540	-.1100	-.0555	-.0132	.0171	.0352	.0413	.0352	.0171	-.0132	-.0555	-.1100

TABLE A3.1



From 'Moments, Shears & Reactions for Continuous Highway Bridges,' AISC 1966

Guadalupe Rd. Bridge 83-58
RR

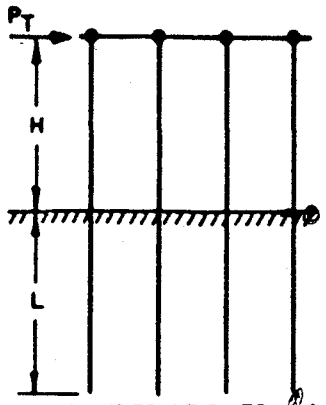
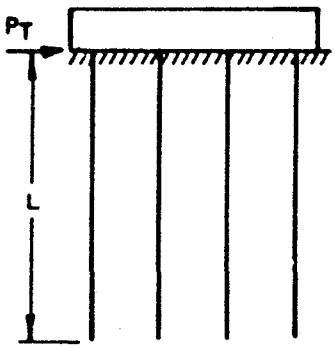
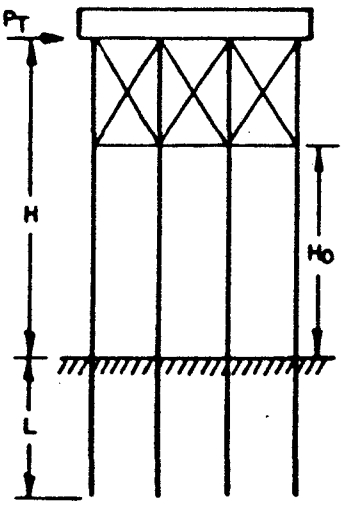
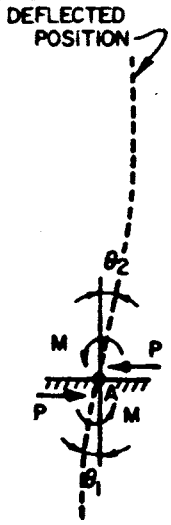
CASE I. FLEXIBLE CAP, ELEVATED POSITION		
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$ DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.
CASE II. PILES WITH RIGID CAP AT GROUND SURFACE		
		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.
CASE III. RIGID CAP, ELEVATED POSITION		
		1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta (\frac{P_T^2}{EI}) + F_\theta (\frac{MT}{EI})$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_0^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

NAVDOCKS DM-7

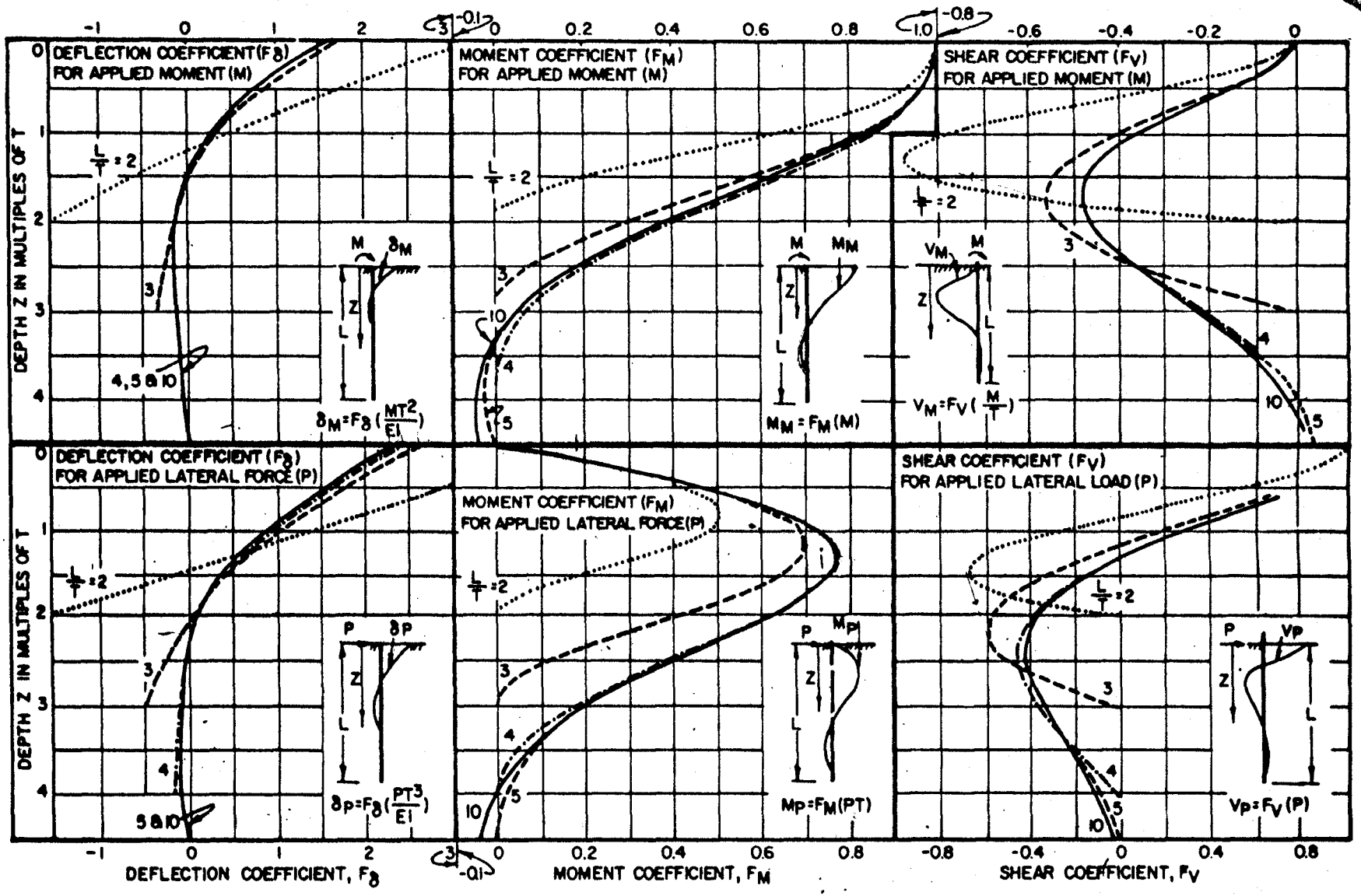


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

NAVDOCKS DM-7

I BASIC PRINCIPLES OF ANALYSIS

A. To behave as a flexible pile, the mechanism of failure must be such that as the failure load is reached, the moment resistance of the pile section is reached before the soil in front of the pile fails in passive pressure or in bearing

B. Flexible action is determined by the relative flexibility of the pile and the stiffness of the soil. There is no way to predict flexible action without actually analyzing the pile. However the following guidelines may be used.

1. If the pile length exceeds 10 pile diameters - flexible action very likely

2. If "rigidity factors" described below exceed the following limits: (See EG-4 for notation) (L = length of pile in inches, EI = modulus and inertia of pile psi·in^4)

Type of Soil →	Clay	Sand
"Rigidity Factor"	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$	$\eta = \sqrt[5]{\frac{n_h}{EI}}$
Flexible action if:	$\beta L > 2.25$	$\eta L > 2.0$
Example	12"ϕ Steel Pipe Pile 20' long EI = $3 \times 10^7 \times 361.5$	
	k_h = 80 pci (stiff clay)	n_h = 10 pci
	$\beta = \sqrt[4]{\frac{80 \times 12}{4 \times 108.45 \times 10^8}} = 0.0120$	$\eta = \sqrt[5]{\frac{10}{3 \times 10^7 \times 361.5}} = 0.0156$
	$\beta L = 0.0120 \times 240 = 2.88 > 2.25$ Flexible Action	$\eta L = 0.0156 \times 240 = 3.74 > 2.0$ Flexible Action

(A convenient table of roots is available in: Abbett - American Civil Eng'g Practice 1956 Vol 1 p 1-1

II ULTIMATE ANALYSIS OF FLEXIBLE PILES

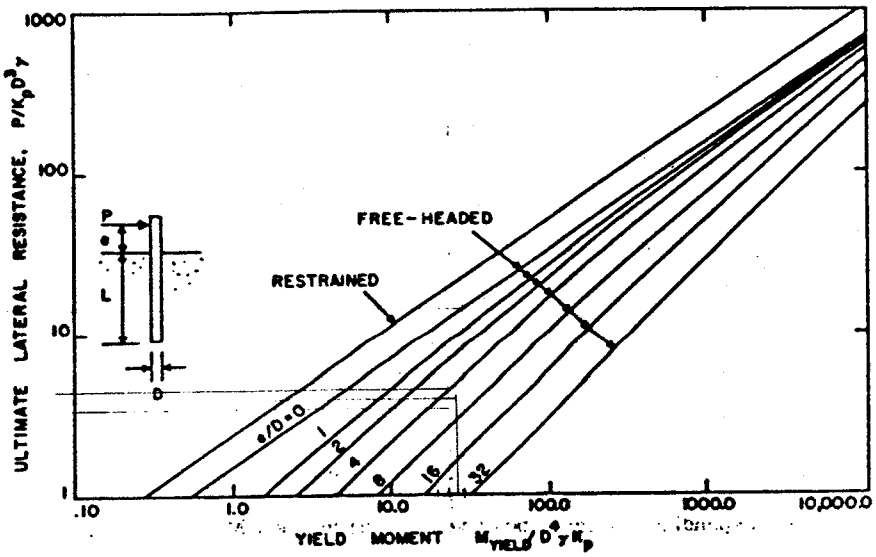
A. Analysis consists of computing the maximum bending moment producing failure in the pile section. Then the ultimate lateral load which produces the failure moment corresponds to the ultimate capacity of the pile. The margin of safety for a particular load is the ratio of the bending moment produced by the load to the resisting moment of the pile section.

B. The bending resistance of piles can be computed by structural analysis methods using elastic or plastic (ultimate) analyses methods. If the pile wall is thin, buckling may become critical

C. If the lateral pressure applied by the pile to the soil exceeds the passive resistance or the bearing capacity of the soil before the full bending resistance is reached, the use of "flexible pile analysis" is invalid.

Broms Method

E. Design charts for Ultimate Analysis of Flexible Piles: (from Ref 5) These charts are based on the equations developed in Ref 3 and 4



-ULTIMATE LATERAL RESISTANCE FOR COHESIONLESS SOILS RELATED TO YIELD MOMENT

Example:

12"ϕ Pipe Pile 20 long
What is max P? e = 0
Soil = sand ϕ = 30° K_p = 3
K_p = 3 γ = 100 pcf

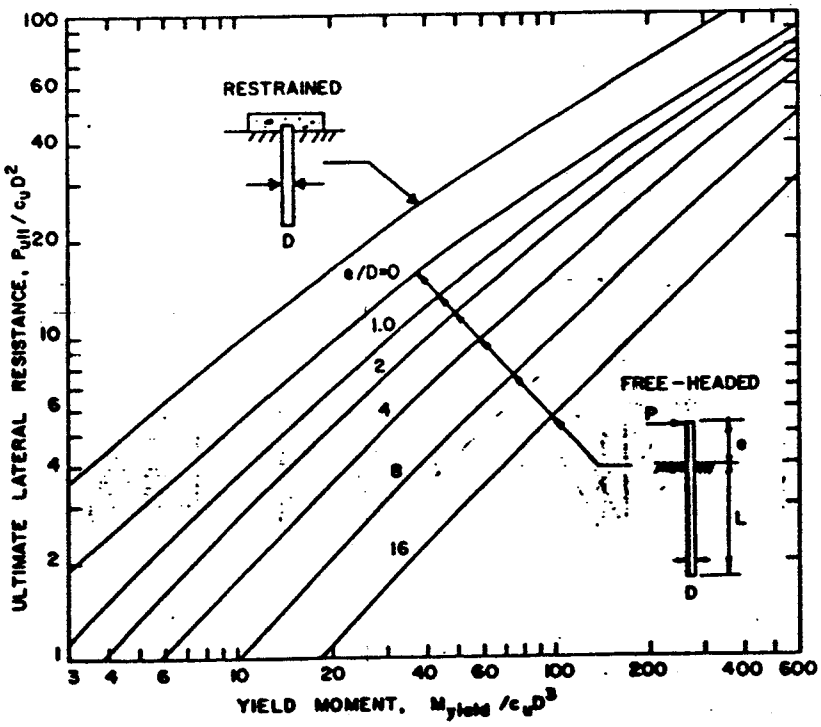
$M_{resisting} = \frac{fI}{c} = \frac{20^* \times 361.5}{6_{in} \times 12 \text{ in/ft}}$
= 100 ft-kips

$\frac{M_r}{D^4 \cdot \gamma \cdot K_p} = \frac{100}{1^4 \times 0.1 \times 3} = 333.3$ ↑ go to chart

$\frac{P}{K_p D^3 \gamma} = 80$ (from chart)

$P = 80 \times 3 \times 1 \times 0.1 = \underline{24 \text{ kips}}$

* allowable yield stress of steel in ksi



-ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO YIELD MOMENT

Example:

Same pile as before c_u = 750 psf

$\frac{M_r}{c_u D^3} = \frac{100}{0.75 \times 1} = 133.3$

$\frac{P_{ult}}{c_u D^2} = 35$

$P_{ult} = 35 \times 0.75 \times 1^2 = \underline{26.25 \text{ kips}}$

Brom's Method

I BASIC PRINCIPLES OF ANALYSIS

A. To behave as a rigid pile, the mechanism of failure must be such that as the failure load is reached, the ultimate bearing capacity or the the passive resistance of the soil is reached before the pile fails in bending.

B. Rigid action is determined by the relative flexibility of the pile and the stiffness of the soil. There is no way to predict rigid action without actually analyzing the pile. However the following guidelines may be used:

1. If pile length is less than 10 pile diameters – rigid action very likely
2. If "rigidity factors" described below are smaller than indicated limits: (See EG-4 for notation) (L = pile length in inches) (E & I Modulus and Inertia of pile – psi and in^4)

MAKE SURE YOU HAVE CONSISTENT UNITS

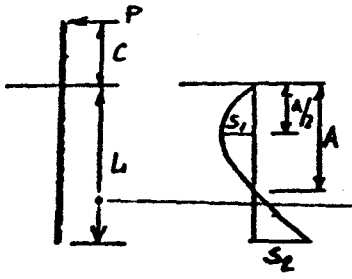
Type of Soil →	Clay	Sand
"Rigidity Factor"	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$ $\frac{\#}{in^3}$ in / (PILE $\frac{\#}{in^2}$ × in^4) = $\sqrt[4]{\frac{1}{in^4}}$	$\eta = \sqrt[5]{\frac{n_h}{EI}}$ – PILE DEGRADATION
"Rigid action" if	$\beta L < 2.25$ (Ref 3)	$\eta L < 2.0$ (Ref 4)
Example	$k_h = 80$ pci (Firm clay) $D = 24''$ $L = 12'$ Concrete $\beta = \sqrt[4]{\frac{80 \times 24}{4 \times 3 \times 10^6 \times 16286}} = 0.010$ $\beta L = 0.010 \times 144 = 1.44 < 2.25$ Rigid Pile action	$n_h = 10$ pci 12" ϕ Steel Pipe Pile $L = 10'$ $\eta = \sqrt[5]{\frac{10}{3 \times 10^7 \times 361.5}} = 0.01563$ $\eta L = 0.01563 \times 120 = 1.87 < 2.0$ Rigid Pile Action

(A convenient table of roots is available in: Abbett – American Civil Eng'g Practice 1956 Vol I p 11-21

II ULTIMATE ANALYSIS OF RIGID PILES

A. Analysis consists of computing the pressure applied by the pile to the soil. Near the surface of the ground (D = 2–3 diameters) a passive failure will occur. Deeper, a bearing failure is expected. The margin of safety available is the ratio of the available resistance to the applied pressure

B. Pressure applied by the pile usually computed by "flagpole formulas"



$$A = \frac{L(3L + 4C)}{4L + 6C}$$

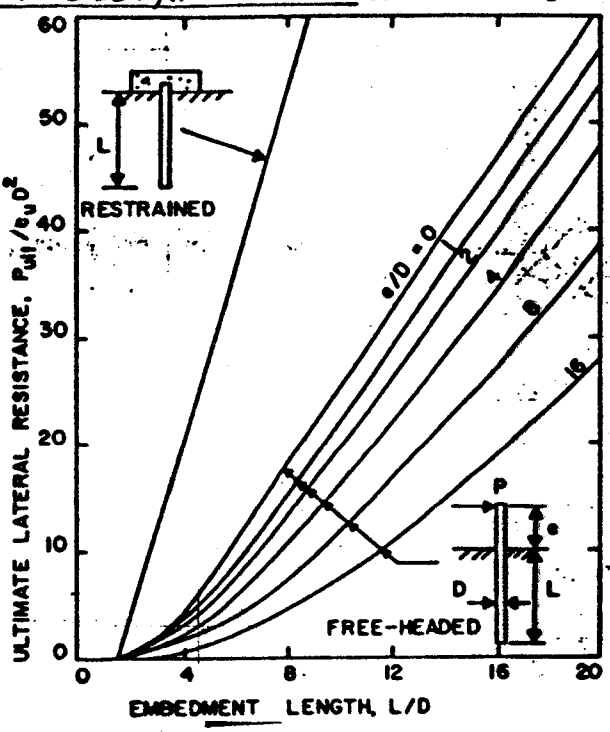
$$S_1 = \frac{1.5P(3L + 4C)^2}{L^2(4L + 6C)}$$ (usually governs design)

$$S_2 = \frac{6P}{L^2}(2C + L)$$

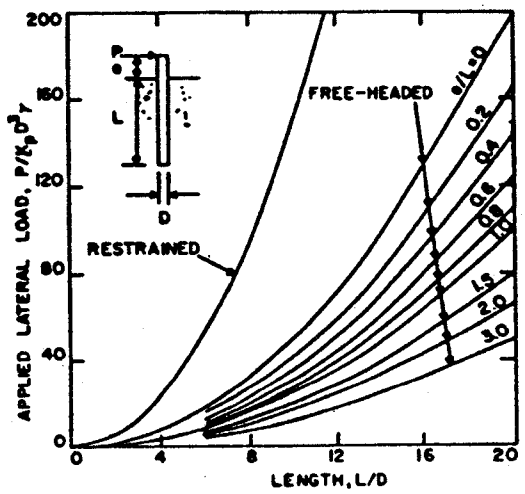
Additional equations for pile design available in Ref 1

C. If bending moments developed in pile exceed strength of the pile the use of "rigid" pile analysis is invalid. Broms Method

G. Design Charts for Ultimate Analysis of Rigid Piles: (From Ref 4)



-ULTIMATE LATERAL RESISTANCE FOR COHESIVE SOILS RELATED TO EMBEDMENT LENGTH



-ULTIMATE LATERAL RESISTANCE FOR COHESIONLESS SOILS RELATED TO EMBEDMENT LENGTH

Example

$c = 750$ psf

$e = 0$ $\quad D = 2'$

$P = 30$ kips

$$\frac{P}{c_u D^2} = \frac{30000}{750 \times 4} = 10$$

$\frac{L}{D} = 5.5 \quad L = 11'$ (FS = 1.0)

Example:

$\phi = 30°$, $D = 1'$, $\gamma = 100$ pcf

$e = 0$, $P = 30$ kips.

$$k_p = \tan^2(45 + \phi/2) = 3.0$$

$$\frac{P}{k_p D^3 \gamma} = \frac{30000}{3 \times 1 \times 100} = 100.0$$

$\frac{L}{D} = 14 \quad L = 14'$ (FS = 1.0)

III BASIC RELATIONSHIPS

Since a rigid pile fails by applying a pressure to the soil which exceed its passive resistance or its bearing capacity, the following is basic:

SOIL FAILS NOT PILE SO DEEPEN DONT INCREASE SIZE

A. Increasing the length of the rigid pile increases the load it can carry and reduces its lateral deflections

B. Increasing the stiffness of the rigid pile will not increase its lateral load carrying capacity and has only a small effect on its deflections.

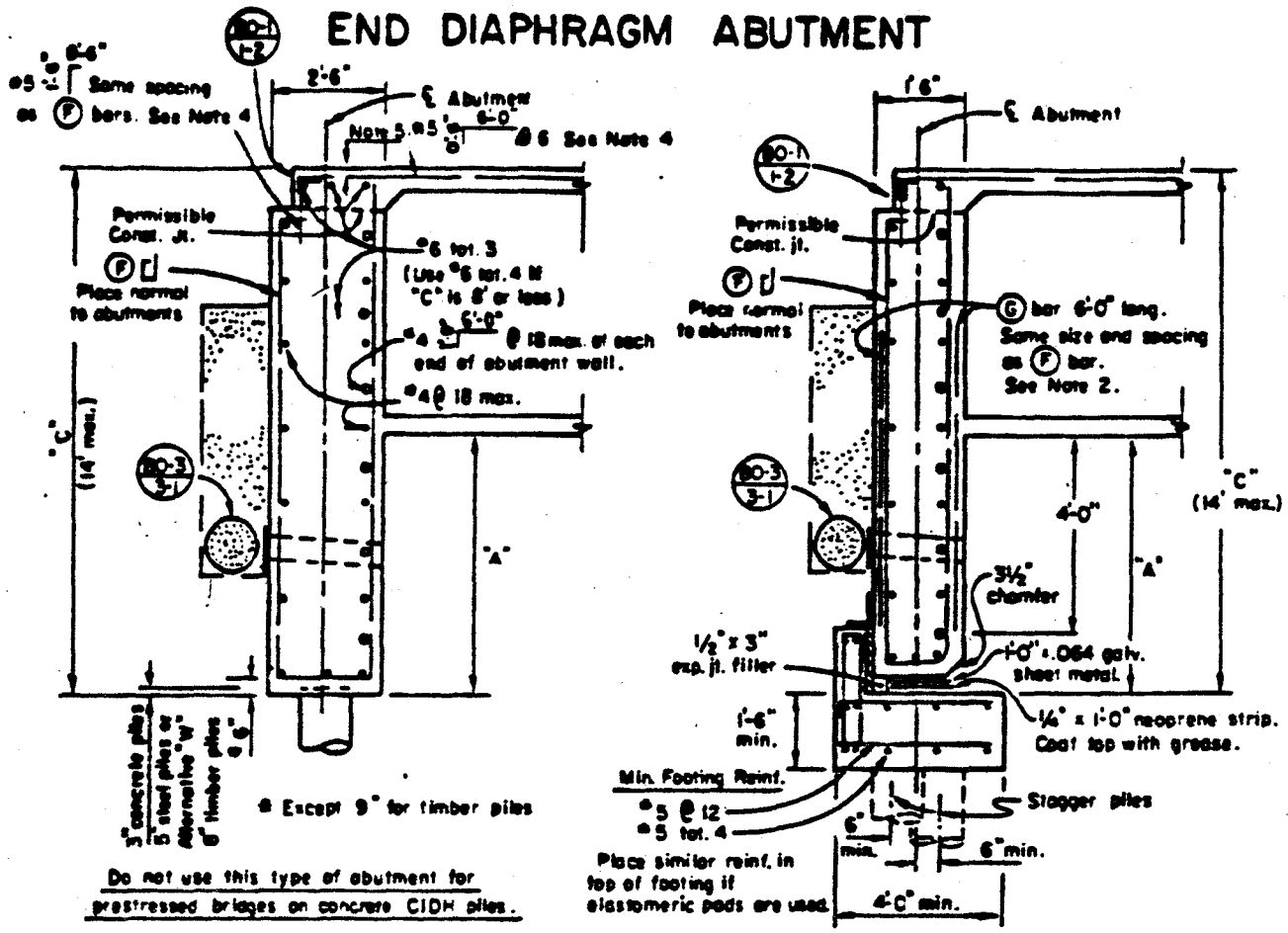
IV ELASTIC ANALYSIS OF RIGID PILES

A. Purpose: to compute deflections of a rigid pile under working loads.

B. Elastic analysis is based on soil stiffness in the elastic range. For this reason it is valid only up to about 1/3 to 1/2 of ultimate load.

C. Soil stiffness values are obtained as described in EG-4 Broms Method

END DIAPHRAGM ABUTMENT



(F) BARS, #5 When (F) bars > than #8

LONGIT. FORCE IN KIPS / L.F.		1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
"A" in Feet	9	#5 @ 18	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#10 @ 12	#10 @ 12	#11 @ 12	#11 @ 12
	8	#5 @ 18	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12	#11 @ 12	#11 @ 12
	7	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12	#10 @ 12
	6	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#8 @ 12	#8 @ 12	#9 @ 12	#9 @ 12	#10 @ 12
	5	#5 @ 18	#5 @ 18	#5 @ 12	#6 @ 12	#7 @ 12	#7 @ 12	#8 @ 12	#8 @ 12	#9 @ 12
	4	#5 @ 18	#5 @ 18	#5 @ 12	#6 @ 12	#6 @ 12	#7 @ 12	#7 @ 12	#8 @ 12	#8 @ 12

NOTES

1. Designer will determine longitudinal force based upon Memo to Designers 5-2. To determine longitudinal force in kips/ft, divide total force by effective abutment width. Effective abutment width = total width less widths of utility openings.
2. Omit G bar when diaphragm width at girder soffit is 2'-6" or greater. Between face of exterior girder and edge of deck extend the G bar upward into the deck slab.
3. When bridge is to be post-tensioned, call for placing of wingwall concrete to be done after stressing is completed.
4. See page 8-32 for requirements on skews.
5. Specify clearance to allow for deck slab reinforcement.
6. Specify keys similar to that shown on page 6-23.

Guadalupe Rd Br 83-58
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