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Design Submittal

POWER 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

POWER ROAD BRIDGE OVER RWCD FLOODWAY

Item No.	Description	Unit	Quantity	Unit Cost	Cost
350	Removal of Existing Improvements	L.S.	1	$15,000	$ 15,000
415	Bridge Guard Rail	L.F.	60	$15	900
420	Chain Link Fence	L.F.	226	$20	4,520
502-1	24"Ø Drilled Shaft	L.F.	754	$40	30,160
502-2	30"Ø Drilled Shaft	L.F.	1,260	$50	63,000
505-1	Concrete f'c=4500 psi	C.Y.	1,113	$200	222,600
505-2	Concrete f'c=4000 psi	C.Y.	596	$180	107,280
505-3	Concrete f'c=3000 psi	C.Y.	122	$150	18,300
505-5	Reinforcing Steel	Lbs.	402,200	$0.35	140,770
	TOTAL COST				$602,530

Cost/SF = $33.80/SF

(SF = 225.7' x 79')

Cannon & Associates, Inc.
Consulting Engineers
Tucson, Arizona

October 9, 1984

Draft Copy
CONSTRUCTION SPECIAL PROVISIONS
FOR
POWER ROAD BRIDGES
AT RWCD FLOODWAY AND CANAL
CONTRACT FCD 83-28

LOCATION OF THE WORK: This project is located on Power Road at the RWCD Floodway and Canal, 500 feet south of Guadalupe 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 215 - CHANNEL EXCAVATION: The work under this section consists of the excavation for the proposed floodway channel in accordance with the plans and Section 215 of the Uniform Standard Specifications.

The excavated material shall be used for roadway fill as shown on the plans.

Volumes will be computed by the average end area method.

Payment for all work under this section will be made at the unit price bid per cubic yard for CHANNEL EXCAVATION.

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 350 - REMOVAL OF EXISTING IMPROVEMENTS: The work under this section consists of the removal and disposal of canal bridge, floodway bridge, existing pavement, detour road and any obstacle to construction, whether shown on the plans or not, unless specifically called out on the plans to be removed or relocated by other agencies.

If a Maricopa County landfill is selected for disposition of road construction waste and/or debris, a Maricopa County Landfill Use Permit will be required. Application for permit can be made at the Maricopa County Landfill Office, located at 3325 West Durango Street, Phoenix, Arizona, 85009, (Telephone 269-2661). Charges

will be levied on a volume basis for each load delivered to the landfill in accordance with the current fee schedule (see attachment). The nearest County landfill to this project is Queen Creek, located at Hawes Road, 0.5 miles south of Chandler Heights Road.

Removal of the canal bridges shall be coordinated with the RWCD to determine the parts that they wish to salvage.

The work under this section also consists of removing the existing barbed wire fence right station 218 on Power Road prior to construction of the detour. The fence shall be stockpiled and re-erected along the right-of-way line after the detour is removed.

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.

The number and kind of barricades, signs, delineators, barriers and all other traffic control devices shall be subject to approval; however, approval of traffic control devices and the approval of the Contractor's method of application of all traffic control measures, shall not relieve the Contractor of the responsibility of protecting the work, the workmen and the traveling public.

A road closure is not authorized. Two lanes shall be open for traffic at all times and continuous access to adjacent residences and the RWCD Canal maintenance road shall be provided. The Contractor shall provide all necessary traffic control devices and measures for this construction to include the striping and the striping removal on the detour road. Paint striping is not authorized on the permanent pavement which will remain after construction. The County shall stripe the finished roads and bridges. The speed limit for the detour road shall be determined by ball banking the curves. All warning signs shall be standard 48 inches in size and mounted or wired down. A 150-foot line of vertical panels with lights and spaced at 30-foot intervals shall be placed on channels 2 feet of the edge of pavement leading to each initial approach to the detour road curve and to each approach to the detour bridge. Traffic shall not be placed on the detour road until all the detour road construction is completed and all traffic measures are installed.

The Contractor shall provide a detailed traffic control plan for approval at the pre-job conference. The plan shall show all measures to include types of signs and barricades and their placement. The Contractor shall designate an employee qualified in construction traffic control to be responsible for implementing, maintaining, inspecting and altering, as necessary, all traffic control measures for construction. A night time telephone number shall be provided. For planning purposes, some of the required measures needed are: road closed and detour signing (see figure 602 of the Manual on Uniform Traffic Control Devices), reduced speed signing, wide load restrictions and all striping to include edge lines and the necessary existing striping removal.

The Contractor shall install and maintain decleration sand berms (approximately five feet high and twenty-eight feet wide) on Baseline Road in the path of through traffic prior to bridge excavation or construction. Sand berms shall remain until road is open to traffic or construction presents less of a hazard than the berms.

All necessary signs and barricades shall remain three working days beyond the acceptance of the project by the District.

Cost for providing and maintaing traffic control shall be indicated to the contract and no additionap payment will be allowed. Such costs shall include all traffic control devices and traffic control measures, such as warning and control signs, barricades, lighting devices, paint striping, delineators, flagmen and other appurtenant items related to traffic control and safety. Included in the cost shall be the maintenance of the detour pavement surface for the full length of the project, including prompt repair of all bumps and holes, and pavement striping of the detour and the finished roadway.

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 506 - PRECAST PRESTRESSED CONCRETE MEMBERS: The work under this section consists of furnishing and placing prestressed concrete beams for the canal bridge in accordance with the plans and Section 506 of the Uniform Standard Specifications.

Payment for all work under this section will be made at the unit price bid for ITEM NO. 506 - PRECAST CONCRETE BEAM, which price shall be full compensation for the item complete in place, including concrete and 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.

The connections shall conform to Section 770 of the Uniform Standard Specifications.

The deck units are stored at the Maricopa County Avondale yard, located on Van Buren Avenue, east of Dysart Road.

The Contractor will select nine of the units, rehabilitate and transport them to the project site, erect them and after the detour road is removed, return them to the Avondale Yard.

Furnishing, placement, removal and salvage of the steel pile bents shall be by the Contractor. The bents shall become property of the Contractor.

Payment for all work under this section will be made at the unit price bid for ITEM NO. 515 - DECK UNITS, which price will be full compensation for the work entailed.

SECTION 525 - CANAL LINING AND IRRIGATION DITCH LINING: The work under this section consists of replacing the canal lining in accordance with the plans and Section 525 of the Uniform Standard Specifications.

Also included are those areas affected by the Detour Bridge pile bents and bypass pipe.

Canal lining shall be 3" thick, pneumatically placed mortar, reinforced with wire mesh (W 1.4 X W1.4-6" x 6").

SECTION 621 - CORRUGATED METAL PIPE, TEMPORARY: The work under this item shall consist of transporting, placing and salvage of corrugated metal pipe of the size and at the locations as called for on the plans. The necessary quantity of pipe, collars and bends are available at the Avondale Yard of the Maricopa County Highway Department, where they must be picked up by the Contractor for transporting to the job site.

At the completion of the project, all corrugated metal pipe shall be removed for salvage and, if deemed reusable by the Engineer, returned to the Avondale Yard. Any pipe deemed not reusable shall be disposed of by the Contractor.

Payment for all work under this section will be made at the unit price bid for CORRUGATED METAL PIPE, TEMPORARY at the specified size, which price will be full compensation for the work entailed.

SECTION 622 - REINFORCED CONCRETE PIPE: The work under this item shall consist of furnishing and placing Rubber Gasket Reinforced Concrete Pipe of the size and at the locations as called for on the plans and conforming to the requirements of Section 735 of the Uniform Standard Specifications. Cost shall include headwall removal and all work necessary to connect the pipe to the existing RCP.

CONTRACT TIME: The Contractor shall complete all canal removals, and canal work during the period of low flow in the canal and shall complete all work on the project within one hundred fifty (150) calendar days after the date of Notice to Proceed.

Structural Calculations
POWER 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

Loading:

A) Design to Accomodate HS20-44 Loading
B) Calculation based on Service loads : the Working Stress Method
C) Thermal Range: Rise = 30°F
Fall = 40°F
Mean = 70°F
D) Seismic Zone: AASHTO Risk Zone 2
E) Stream Flow: V = 7.0 ft/s, d = 7.2', Q = 4900 cfs
F) Allowance for 25 psf future wearing surface

Materials:

A) Concrete:

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

B) Steel

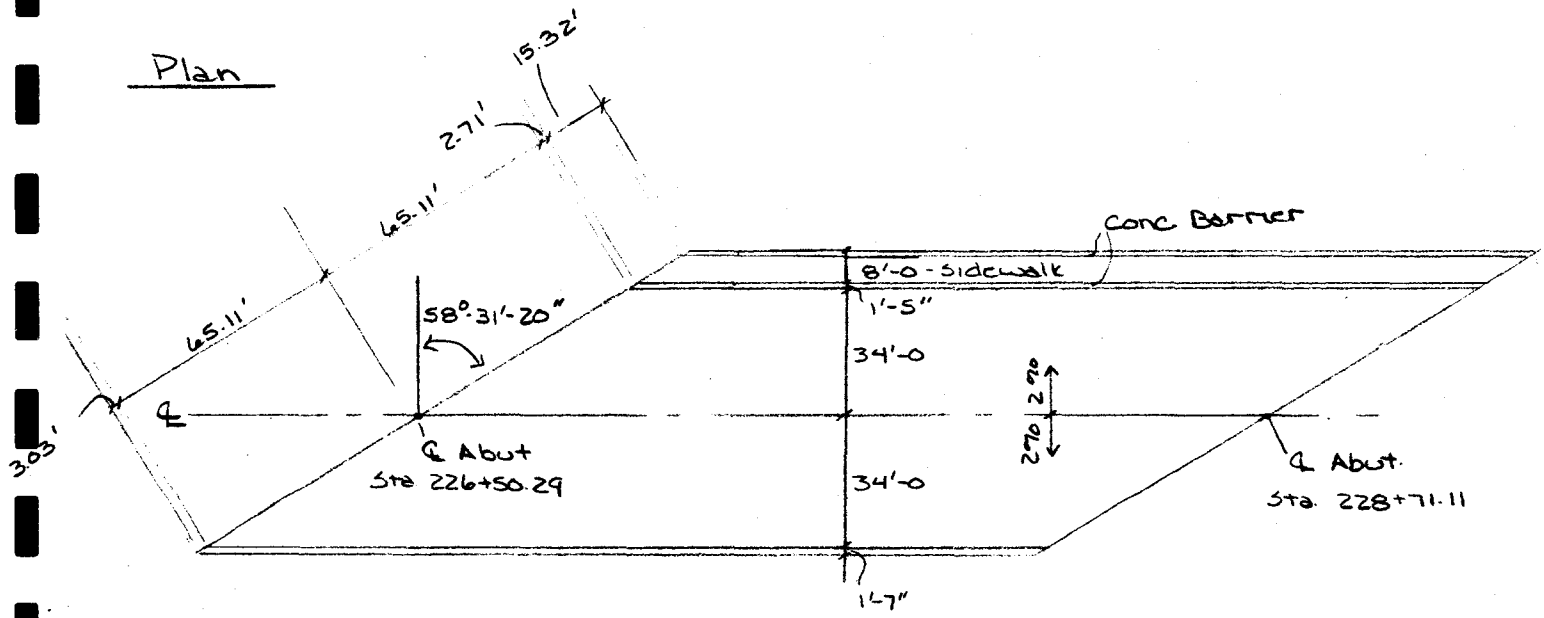
Bars #6 and smaller - Grade 40	f_s = 20,000 psi
Bars #7 and larger - Grade 60	f_s = 24,000 psi

Assumptions:

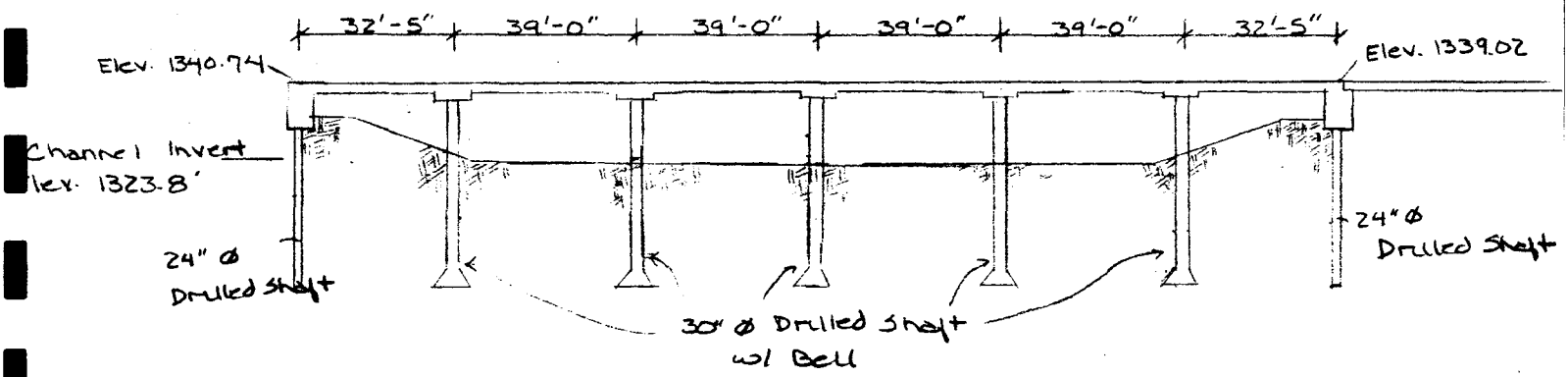
1.) Pier Columns are pinned to girders - slab continuous
2) Integral Abutment - Abutment cap is pinned to caissons
3) Scour Elev. @ 1313.8 (10' below Channel Invert)

Configuration

Plan



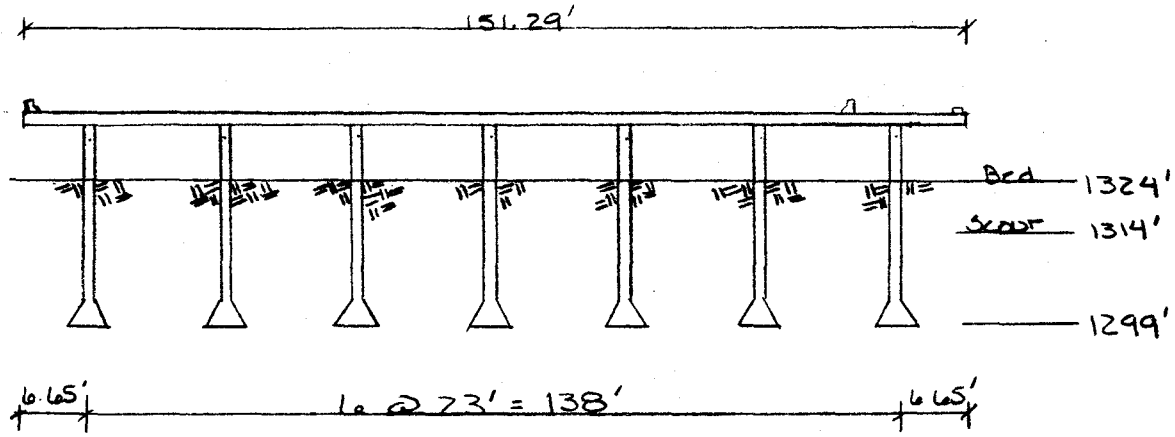
Pier Elevation



Cannon & Associates, Inc.
Consulting Engineers

project Power Rd Bridge 83-58
designed by RR
checked by MRE 10.8.84 RT
date 8/17/84 sh. 4 of

Pier Elevation



Cannon & Associates, Inc.
Consulting Engineers

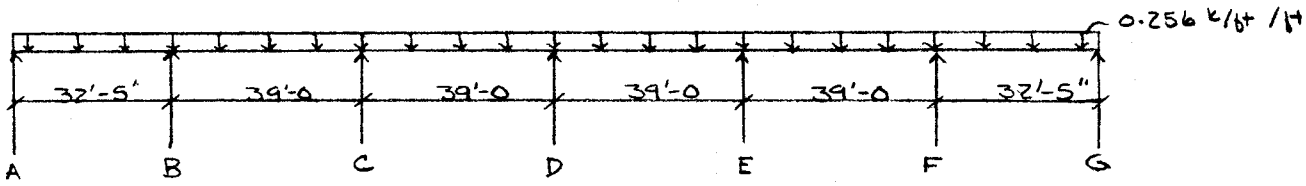
project Power Rd Bridge
designed by RR 83-058
checked by RT 10-4-84 RT
date 8/31/84 sh. 5 of

Deck Design

Deck Design

- Assume Slab Thickness = 18 1/2"
- Reinforcement Parallel to Traffic

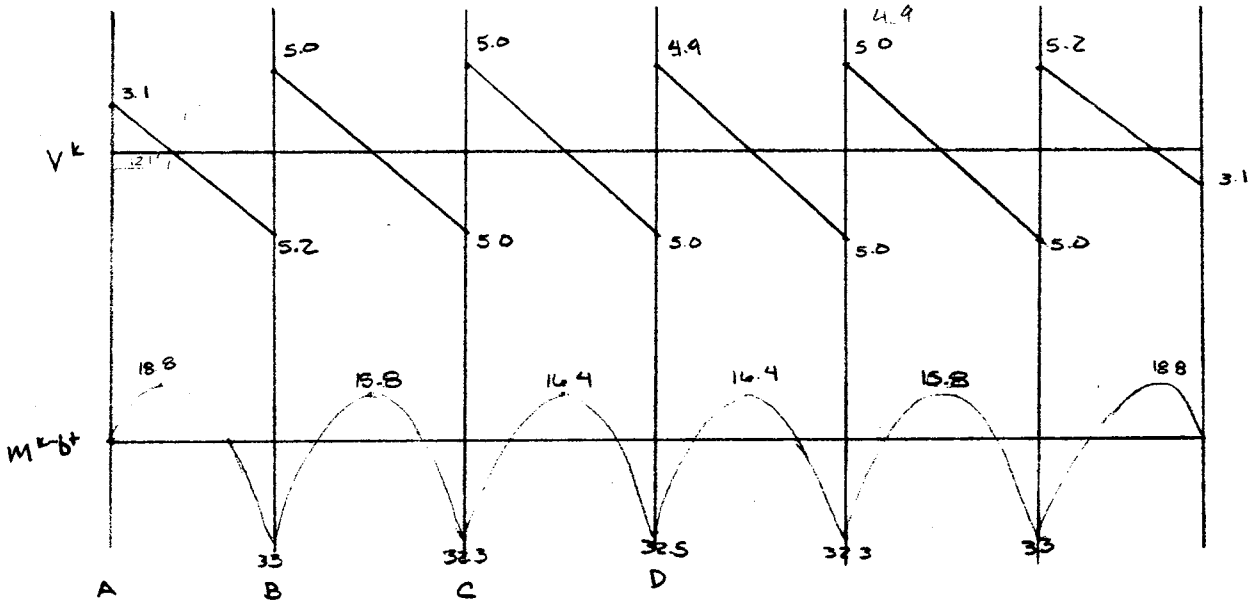
Dead Load: Wearing Surface: = 0.025 ksf
Slab: (.150)(18.5/12) = 0.231 ksf
0.256 ksf



Three Mom Eq.: $M_A l_1 + 2M_B(l_1+l_2) + M_C l_2 = -\frac{w}{4}(l_1^3 + l_2^3)$

$M_A = M_G = 0$; $M_B = M_F = -33.0^{k\text{-}ft}$; $M_C = M_E = -32.3^{k\text{-}ft}$; $M_D = -32.5^{k\text{-}ft}$

$R_A = R_G = 3.1^k$; $R_B = R_F = 10.2^k$; $R_C = R_E = 10.0^k$; $R_D = 9.9^k$



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Live Load:

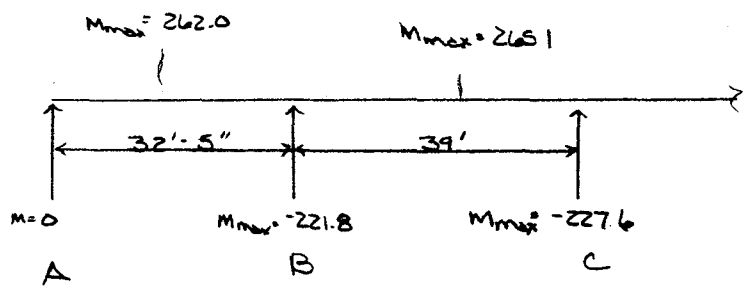
Ref. 'Moments Shears and Reactions For Continuous Highway Bridges'

Assume LL Reactions like those calculated for 4 Spans (Conservative)

N = 39/32.42 = 1.20 - use Table 4.2 - See Appendix

Total Length = 2(32.42 + 39) = 143'

Max. Moments:



Impact Coefficient = 0.300 (AASHTO 1.2.12)

Distribution of Lane Loads (AASHTO 1.3.2(c))

$2E = 2(4 + 0.06(32.42)) = 11.9'$ (32.42' span)

$= 2(4 + 0.06(39)) = 12.7'$ (39' span)

DL + LL + I (Moments/ft. width)

Exterior Span -

DL = 18.8 k-ft

LL + I = 1.3 (262/11.9) = 28.6 k-ft DL + LL + I = 47.4 k-ft

B: DL = -33.0 k-ft

LL + I = 1.3 (-221.8/11.9) = -24.2 k-ft DL + LL + I = -57.2 k-ft

Interior Span -

DL = 16.4 k-ft

LL + I = 1.3 (265.1/12.7) = 27.1 k-ft DL + LL + I = 43.5 k-ft

Support

C: DL = -32.5 k-ft

LL + I = 1.3 (-227.6/12.7) = -23.3 k-ft DL + LL + I = -55.8 k-ft

$\frac{9+10}{30} = \frac{19}{30} = 1.63$ 1-7½ S/Q

Use

Use spacing of bars = 6" ; h = 18½"

Positive Reinforcement **Use #9 @ 6"**

Using f'c = 4000 n = 8

	+ M
DL	20.1
LL+I	28.4
	48.5

M = 47.4 k-ft Try #9 @ 6" (#10 @ 8) $A_s = 2(1.00) = 2.00$ (1.91) in² $A_s = \frac{48.5 \times 12}{24 \times .89 \times 17.3} = 1.57$

d = 19.5 - 1 - 0.7 - 0.5 = 17.3

d = 18½ - 1 - 1.128/2 = 16.9" Use d = 16" n = 29,000/3850 = 7.5 use n = 8 ✓ ok for 4000

$n\frac{A_s}{bd} = 8\frac{2.00}{(12)(16)} = 0.083$ (1.91, 17.3; .074) k = 0.333 (.318) j = 0.889 (.894)

req'd $d = \sqrt{\frac{M}{\frac{1}{2}f_c k j b}} = \sqrt{\frac{47.4(12)}{\frac{1}{2}(1.8)(0.333)(0.889)(12)}} = 13.3" < 16"$ O.K

$f_s = \frac{M}{A_s j d} = \frac{47.4(12)}{(2.00)(.889)(16)} = 20.0$ ksi < 24.0 ksi (48.5; 1.91, .894, 17.3; 19.7)

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.333(20)}{8(1-0.333)} = 1.25$ ksi < 1.8 ksi ∴ O.K (1.17; .318; 1.6)

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$-M$
$DL = 36.1$
$LLF = 25.4$
61.5

Neg. Reinforcement Use #10 @ 6" $A_s = \frac{61.5 \times 12}{24 \times .89 \times 16.8} = 2.06$

$M = 57.2$ k-ft Try #10 @ 6" $A_s = 2(1.27) = 2.54$ in² $d = 18\frac{1}{2} - 2\frac{1}{2} - \frac{1.27}{2} = 15.3''$ Use $d = 15''$

$n\frac{A_s}{bd} = 8\frac{2.54}{(12)(15)} = 0.113$ $k = 0.376$ $j = 0.875$

req'd $d = \sqrt{\frac{57.2(12)}{\frac{1}{2}(1.8)(.376)(.875)(12)}} = 13.9'' < 15''$ O.K

$f_s = \frac{M}{A_s j d} = \frac{57.2(12)}{2.54(.875)(15)} = 20.6$ ksi ≤ 24.0 ksi OK

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.376(20.6)}{8(1-.376)} = 1.55$ ksi < 1.80 ksi ∴ O.K

Development Lengths (AASHTO 1.5.14)

Positive Reinf - #9 bar

$$0.8 l_d = 0.8\frac{0.04(1.00)(60{,}000)}{\sqrt{4500}} = 28.6''$$

Negative Reinf - #10 bar

$$1.4(0.8) l_d = 1.4(0.8)\frac{0.04(1.27)(60{,}000)}{\sqrt{4500}} = 50.9''$$

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

Bottom of Slab:

Distribution Reinforcement $\% = \frac{100}{\sqrt{S}} = \frac{100}{\sqrt{32.42}} = 17.6\%$ (32'-5" span)
$= \frac{100}{\sqrt{39}} = 16.0\%$ (39' span)

$.176\left(\frac{20.0}{24.0}\right)(2\text{ in}^2) = 0.29\text{ in}^2/12\text{ in}$ use #5 @ 9" $= .41$

Top of Slab:

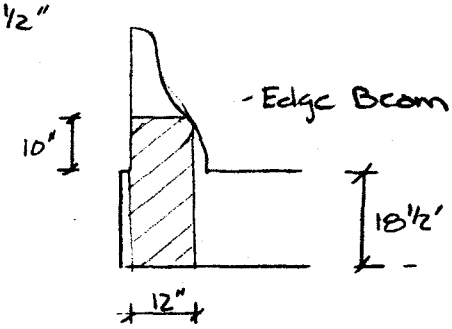
Distribution Reinforcement:

Use #4 @ 18" $A_s = 0.133\ in^2/ft$

Edge Beam Design

width of beam = 1'-0" depth = 18½" + 10" = 28½"

Dead Load: Slab 0.231 k/ft
Barrier 0.40 k/ft
0.631 k/ft



DL Moments (From Deck Calc)

Exterior Spans (@ 0.4 AB) $M = \frac{0.631}{0.256}(18.8) = 46.3$ k-ft
Supports $M = \frac{0.631}{0.256}(-33.0) = -81.3$ k-ft
Interior Spans $M = \frac{0.631}{0.256}(16.4) = 40.4$ k-ft

Live Load (AASHTO 1.3.2 (D))

Live Load Moment of 0.10 PS P = 16k

Max + Mom. @ 0.40 Exterior Spans:
Table A4.2 'Moments Shears & Reactions ....'
Load @ 0.4 AB Coeff = 0.2070
$M = 0.2070(16)(32.42) = 107.4$ k-ft
$0.1(107.4) = 10.7$ k-ft

Max - Mom @ Support
Table A4.2
Load @ 0.4 Span 2 Coeff = -0.1038
$M = -0.1038(16)(32.42) = -53.8$ k-ft
$0.1(-53.8 \text{ k-ft}) = -5.4$ k-ft

Total Moments:

Max + M = 46.3 + 10.7 = 57.0 k-ft
- M = -81.3 - 5.4 = -86.7 k-ft

Edge Beam (cont)

Design for M = -86.7 k-ft

$d = 28\frac{1}{2} - 3 = 25\frac{1}{2}''$ Use $d = 25''$ $A_s = 2(1.00) = 9.00 \text{ in}^2$

$n\frac{A_s}{bd} = 8\frac{2.00}{(12)(25)} = 0.0533$ $k = 0.278$ $j = 0.907$

Req'd $d = \sqrt{\frac{86.7(12)}{\frac{1}{2}(1.8)(.278)(.907)(12)}} = 19.5'' < 25''$ o.k

$f_s = \frac{(86.7)(12)}{(2.00)(.907)(25)} = 22.9 \text{ ksi} < 24.0 \text{ ksi}$

$f_c = \frac{0.278(22.9)}{8(1-.278)} = 1.10 \text{ ksi} < 1.80 \text{ ksi}$

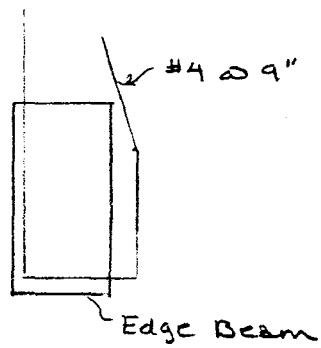
Pos. Moment = 57.0 k-ft Use 2 #9

Shear:

$V_{max} = \left(\frac{0.631}{0.256}\right)(5.2^k) = 12.8^k$

$v = 12.8/(12)(25.5) = 0.042 \text{ ksi}$

$v_c = 0.95\sqrt{4500} = 63.7 \text{ psi} > 42 \text{ psi} > 31.8 \text{ psi}$



AASHTO 1.5.10 Req'd $A_v = \frac{50bws}{f_y}$

#4 @ 9" $A_v = \frac{50(12)(9)}{40,000} = 0.135 \text{ in}^2 < 0.20 \text{ in}^2$ ∴ o.k

NOTE:

THE EXISTING LONGITUDINAL DECK REINF. IS SUFFICIENT TO RESIST THE POSITIVE MOMENT IN THE EDGE BEAM.

Pier Design

Pier Cap Design

Bridge Skew = 58°-31'-20" ℓ = 151.3'

From Deck Calc:

R_{DL} = 10.2 k/ft Due to Skew: w_{DL} = cos 58.52° (10.2) = 5.33 k-ft

Use cap d = 7½"; w = 4'-0"

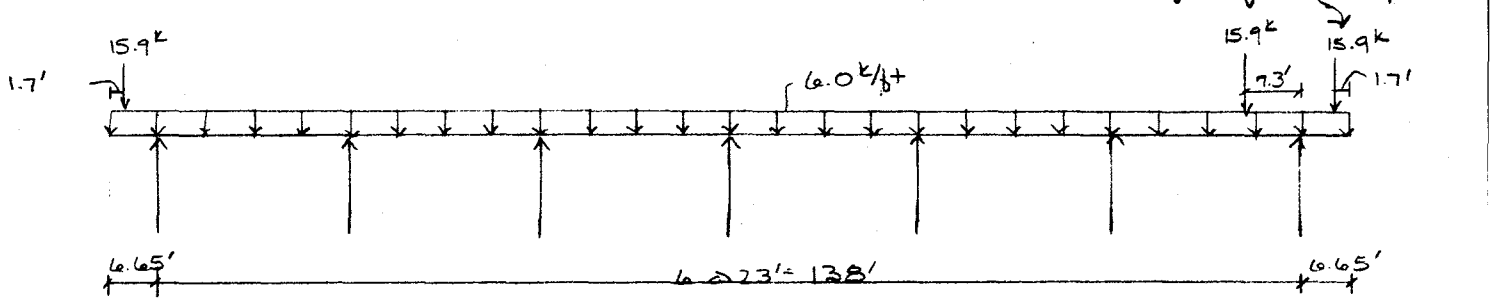
w_D = 5.33 + .150(4)(7.5/12) = 5.7 k/ft Use w_D = 6.0 k/ft

Load from Barrier - 0.4 k/ft

Concentrated Load = $\frac{0.4}{0.256}$(10.2) = 15.9 k

Note: Barrier rather than handrail load used to account for fu. lane potential

Dead Load



211.4	264.5	264.5								-289.6	318.6	-211.4
	53.1	26.5								-53.6	-107.2	
	-211.4	291.0								-343.2	211.4	
		3/7	4/7	1/2	1/2	1/2	1/2	1/2	1/2	4/7	3/7	
	-211.4	291.0	-264.5	264.5	-264.5	264.5	-264.5	264.5	-264.5	264.5	-343.2	211.4
		-11.4	-15.1	0	0	0	0	0	0	45.0	33.7	
			0	-7.6		0	0	0	22.5	0		
		0	0	3.8	3.8	0	0	-11.2	-11.2	0	0	
			1.9	0	0	1.9	-5.6	0	0	-5.6		
		-.8	-1.1	0	0	1.9	1.9	0	0	3.2	2.4	
	-211.4	278.8	-278.8	260.7	-260.7	268.3	-268.3	253.3	-253.3	307.1	-307.1	211.4

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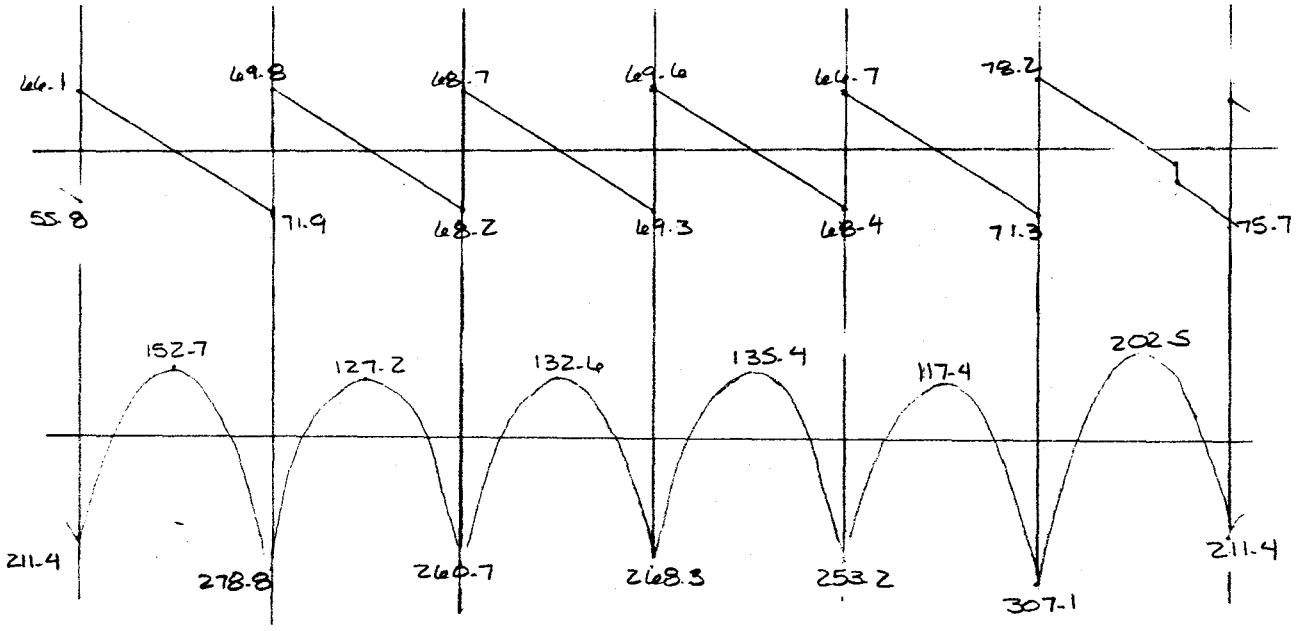
Dead Load Shear & Mom:



Live Load Moments:

Ref.: 'Moments, Shears, & Reactions for Cont Highway Bridges'
Table 4.2 - See Appendix

Reaction/Lane = 64.5^k (Interpolating for L = 26.39 + 32.42) = 142.8'

$w_{LL} = 64.5^k \cos 58.52^\circ / 12' = 2.81\ ^k/ft$

max $\pm M_{LL} \approx 2.81(23)^2/8 = 186\ ^{k\text{-}ft}$

Max Pos. Mom D+L+I Use 10 #9

$M = 152.7^{k\text{-}ft} + 1.3(186^{k\text{-}ft}) = 394.5^{k\text{-}ft}$ ($M_{DL} = 202.5$ cann't occur w/ $M_{LL} = 186$)

Try 10 #9 $d = 26'' - 1\tfrac{1}{2}'' - .625'' - 1.128/2 = 23.3''$. Use $d = 23''$ $A_s = 10.00\ in^2$

$n\frac{A_s}{bd} = 8.0\frac{10.00}{(48)(23)} = 0.0725$ $k = 0.315$ $j = 0.895$

Req'd $d = \sqrt{\frac{394.5(12)}{\frac{1}{2}(1.8)(.315)(.895)(48)}} = 19.7'' < 23''$ ✓

$f_s = \frac{M}{A_s j d} = \frac{394.5(12)}{(10.00)(.895)(23)} = 23.0\ ksi < 24.0\ ksi$

$f_c = \frac{k f_s}{n(1-k)} = \frac{0.315(23.0)}{8(1-.315)} = 1.32\ ksi < 1.80\ ksi$ ∴ O.K

Max Neg Mom D+L+I Use 10 #11

$M = 278.8^{k\text{-}ft} + 1.3(186^{k\text{-}ft}) = 520.6^{k\text{-}ft}$

Try 10 #11 $d = 26'' - 2\tfrac{1}{2}'' - 1.27'' - .5'' - .625'' - 1.41/2 = 20.4''$ $A_s = 10(1.56) = 15.6\ in^2$

Use $d = 20''$

$n\frac{A_s}{bd} = 8.0\frac{15.6}{(48)(20)} = 0.130$ $k = 0.396$ $j = 0.868$

Req'd $d = \sqrt{\frac{520.6(12)}{\frac{1}{2}(1.8)(.396)(.868)(48)}} = 20.5'' > 20''$

∴ Treat as Doubly Reinforced

$f_s = 24\ ksi$ $f_c = 1.8\ ksi$ $d = 20.$ $d' = 1\tfrac{1}{2}'' + .625'' + 1.128/2 = 2.69''$ Use $d' = 3''$

$k = \frac{D}{n+r}$ $r = f_s/f_c = 24/1.8 = 13.33$ $k = \frac{8}{8+13} = 0.381$ $j = 0.873$

$M = M_1 + M_2$

Tensile Steel $A_s = A_{s1} + A_{s2}$; Comp. Steel $= A_s'$

$M_1 = \frac{f_c}{2} k j b d^2 = \frac{1.8}{2}(0.381)(0.873)(48)(20'')^2 = 5747$ k-in $= 479$ k-ft

$A_{s1} = \frac{M_1}{f_s j d} = \frac{479(12)}{24(.873)(20)} = 13.7$ in^2

$M_2 = M - M_1 = 520.6 - 479 = 42$ k-ft

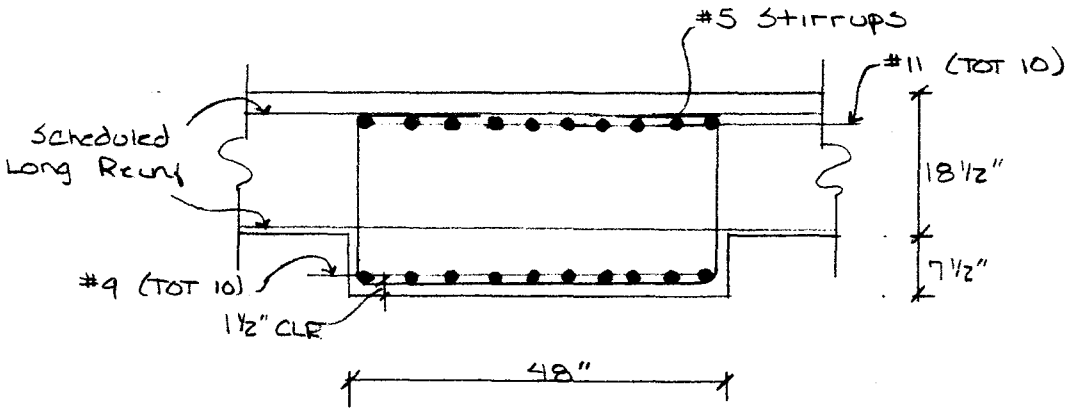
$M_2 = A_{s2} f_s (d - d') = A_s' f_s' (d - d')$

$f_s' = 2 f_s \frac{k - d'/d}{1-k} = 2(24) \frac{.381 - 3/20}{1 - .381} = 17.9$ ksi

$A_{s2} = \frac{42(12)}{24(20-3)} = 1.24$ in^2 $\qquad A_s' = \frac{42(12)}{17.9(20-3)} = 1.66$ in^2

<u>Tensile Reinf</u> $A_s = 13.7 + 1.24 = 14.9$ in^2 $\qquad$ <u>use 10 #11</u> $A_s = 15.6$ in^2

<u>Comp Reinf</u> $A_s' = 1.66$ in^2 $\qquad$ 2 #9



<u>Pier Cap Section</u>

Shear Reinforcement

@ dist. d (20") from Interior Support:

V_{max} DL = $71.9^k - 6.0(20/12) = 61.9^k$

LL: V_{max} LL = $(2.81^k/ft)(23')/2 - 2.81(20/12) = 27.6^k$

$V_{D+L+I} = 61.9^k + 1.3(27.6^k) = 97.8^k$

$v = \frac{V}{bd} = \frac{97.8^k}{(48")(20")} = 0.102$ ksi $v_c = 0.95\sqrt{4500} = 63.7$ psi

#5 stirrup $A_v = 2(0.31) = 0.62$ in^2

Req'd $S = \frac{0.62(20{,}000)}{(102-63.7)(48)} = 6.7"$ Use $S = 6"$

Max Spacing of #5 Stirrups $S = \frac{0.62(40{,}000)}{50(48)} = 10"$

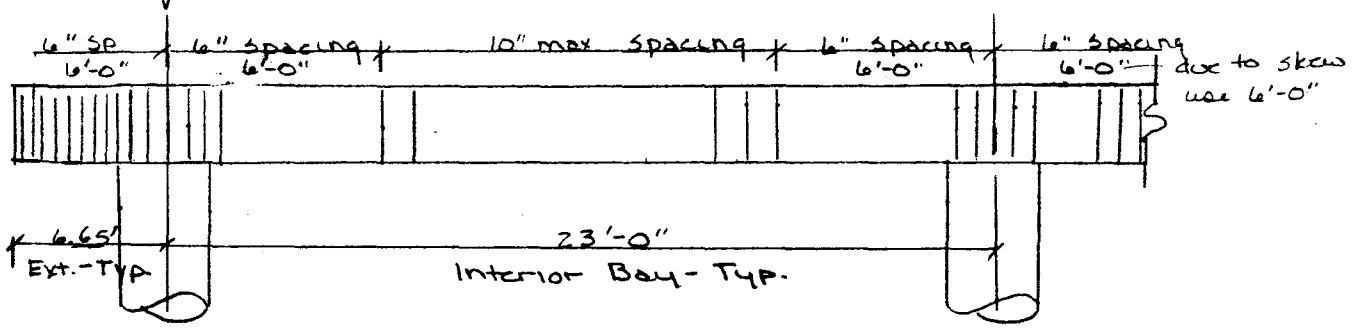
3' from Support:

$V_{DL} = 71.9 - 6.0^k(3') = 54^k$

$V_{LL} = (2.81)(23/2 - 3) = 24^k$

$V_{D+L+I} = 54 + 1.3(24) = 85.2^k$ $v = \frac{85.2}{(48)(20)} = 0.089$ ksi

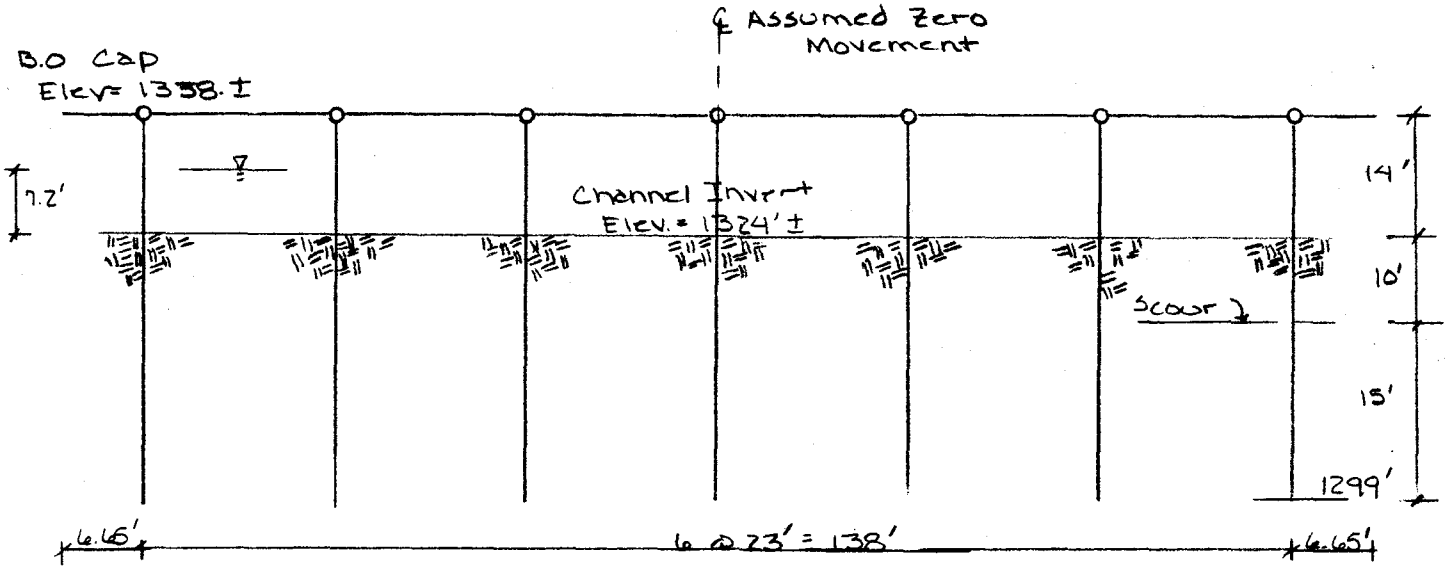
Req'd $S = \frac{0.62(20{,}000)}{(89-63.7)(48)} = 10.2"$ Use $S = 10"$



Pier Column

- Embedment depth of piles checked using Brom's Method for the cohesionless and cohesive soil cases. - See Appendix
- Columns Designed using NAVDOCKS DM-7 - See Appendix

Transverse Model



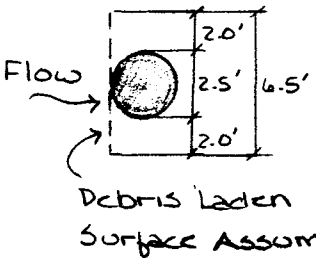
Stream Flow Forces

(AASHTO 1.2.17) $P = KV^2$

$V = 7.0$ ft/s $K = 1.375$

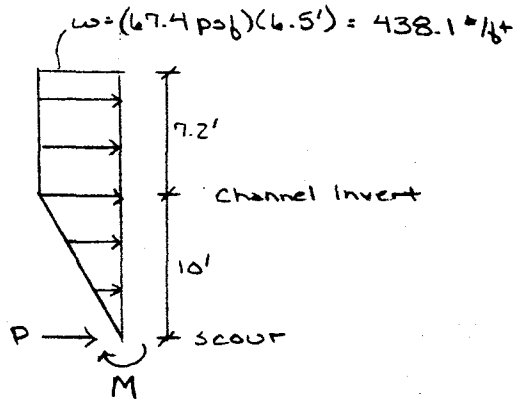
$P = 1.375(7.0)^2 = 67.4$ psf

Depth of Flow = 7.2'



Per Corps of Engrs. Recomm.

Stream Flow Forces (cont.)



$P = 438.1(7.2) + 438.1(10)/2 = 5.34^k$

$M = 438.1(7.2)(10 + 7.2/2) + 438.1(10/2)(2(10)/3) = 57.5^{k\text{-}ft}$

Temperature Forces

Transverse Direction:

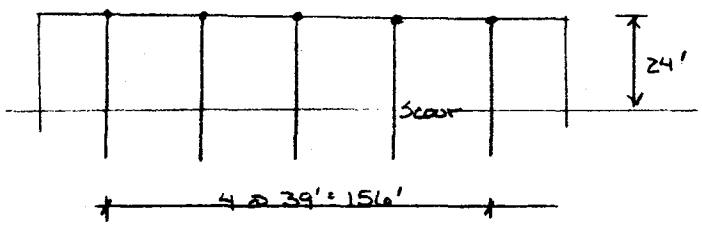
$\Delta_{\substack{outside \\ Pier}} = (0.000006 \times 69')(12)(40°F) = 0.20''$

$P = \frac{3EI\Delta}{L^3}$ $E = 3122$ ksi $I = \pi(15)^4/4 = 39{,}761 \text{ in}^4$ $L = 24'$

$P = \frac{3(3122)(39{,}761)(.20)}{(24 \times 12)^3} = 3.12^k$

$M_{@\,scour} = 3.12^k(24') = 75^{k\text{-}ft}$

Longitudinal Direction:

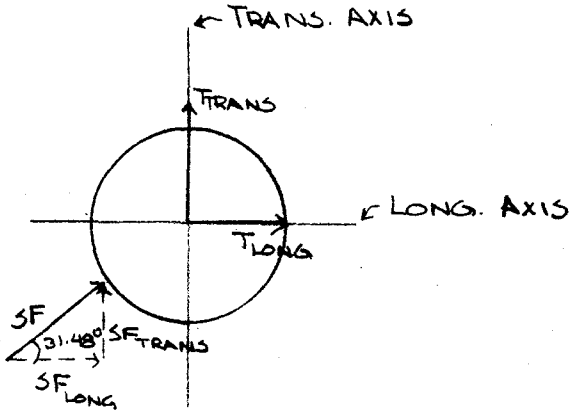


$\Delta_{\substack{outside \\ Pier}} = (0.000006)(78')(12)(40°F)$
$= 0.22''$

$P = \frac{3(3122)(39761)(0.22)}{(24 \times 12)^3} = 3.43^k$

$M_{@\,scour} = (3.43^k)(24') = 82^{k\text{-}ft}$

Load Case IV - Stream Flow and Temperature



$T_{TRANS} = 3.12^k$ @ 24'

$T_{LONG} = 3.43^k$ @ 24'

$SF = 5.34^k$ $\quad SF_{LONG} = 5.34 \cos 31.48° = 4.55^k$ @ 10.8'

$SF_{TRANS} = 5.34 \sin 31.48° = 2.79^k$ @ 10.8'

$P_T = 3.12^k + 2.79^k = 5.91^k$ @ $\dfrac{3.12(24) + 2.79(10.8)}{5.91} = 17.8'$

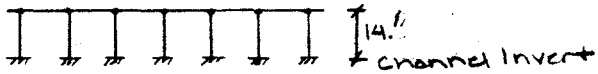
$P_L = 3.43^k + 4.55^k = 7.98^k$ @ $\dfrac{3.43(24) + 4.55(10.8)}{7.98} = 16.5'$

$P = (5.91^2 + 7.98^2)^{1/2} = 9.93^k$ $\quad$ <u>Use $P = 10^k$ @ 17'</u>

$M_{@\ SLOPE} = 10^k(17') = 170^{k\text{-}ft}$

Earthquake Forces (AASHTO 1.2.20)

EQ = CFW



F = 1.0

W = Dead Load =	Wearing Surface	= (.025 ksf)(76')(221')	= 420^k
	Barriers	= (.4 k/ft)(221')(2)	= 177^k
	Slab	= (.150 k/ft³)(18.5/12)(221)(79')	= 4037^k
	Haunches	= (.150 k/ft³)(7.5/12)(4')(151.3')(5)	= 284^k
	Columns	= (.150 k/ft³)(14')(4.91 ft²)(35)	= 361^k
	Pedestrian Curb & Handrail	= (6.2 k/ft)(221')	= 44^k
			5323^k

- Assume piers resist total seismic force

$P = P_1 + P_2$ $\quad$ P_1 - force to cause 1" movement of piers
P_2 - force to cause 1" movement of abutment

$$P_1 = \frac{3EI\Delta}{L^3} \times 35 \text{ piers}; \quad \Delta = 1''$$

$$P_1 = \frac{3(3122)(39.761)(1)}{(14 \times 12)^3} \times 35 = 2749^k$$

P_2 - Calc using NAVDOCKS DM-7 - See Appendix
$f = .03$ ksi $\quad EI = 3122(\pi)(12)^4/4) = 5.08 \times 10^7$ k-in²

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(5.08 \times 10^7 / .03\right)^{1/5} = 70.1''$$

$L = (1340 - 10) - 1299 = 31'$ $\quad L/T = 31(12)/70.1 = 5.3$

$$P_2 = \frac{\delta E I}{F_\delta T^3} \times 13 \text{ shafts/abut} \times 2 \text{ abut} \quad F_\delta = 2.3 \quad \delta = 1''$$

$$P_2 = \frac{1(5.08 \times 10^7)}{2.3(70.1)^3} \times 26 = 1667^k$$

$P = 2749^k + 1667^k = 4416^k$

$T = 0.32\sqrt{W/P} = 0.32\sqrt{5323/4416} = 0.35$

From AASHTO Fig. 1.2.20 D (Zone II A=0.22g) C= 0.085

$EQ = 0.085(1.0)(5323) = 452^k$

- Assume earthquake force is evenly distributed between all 35 columns

$EQ/col = 452/35 = 13^k$

$M_{@\ invert} = 13^k(14') = 182^{k-ft}$

Embedment Depth per Brom's Method

See Appendix

Per Soil Report -

1) Cohesive case: $K_hD = 400\,\#/in^2$ ($K_hD = 200\,\#/in^2$ for Earthquake)
2) Cohesionless case: $n_h = 60\,\#/in^3$ ($n_h = 30\,\#/in^3$ for Earthquake)

Concrete Pile: $D = 30''$; $I = 39{,}761\ in^4$; $E = 3140$ ksi

Soil: $\phi = 30°$; $C_u = 900$ psf ; $K_p = 3.0$; $\gamma = 120\,\#/ft^3$

$L = 15'$ Stream Flow & Temp (Case IV)

$L = 25'$ Earthquake (Case VII)

Cohesive Case:

IV: $\beta = \sqrt[4]{\frac{K_hD}{4EI}} = \sqrt[4]{\frac{400\,\#/in^2}{4(3140000\ psi)(39761\ in^4)}} = 0.0053/in$

$\beta L = 0.0053(15')(12) = 0.95 < 2.25$ ∴ Rigid Pile Action

Stream Flow & Temp: ($L = 15'$)

$P = 10^k$ @ $17'$ $e/D = 17'/2.5 = 6.8$ $\frac{P}{C_uD^2} = \frac{10{,}000}{900(2.5)^2} = 1.78$

Req'd $L/D = 4.5 \pm$ $L = 4.5(2.5) = 11.2 < 15$ $FS = \frac{15}{11.2} \times 1.25 = 1.67$

VII: $\beta = \sqrt[4]{\frac{200\,\#/in^2}{4(3140000)(39761)}} = 0.0045/in$

$\beta L = 0.0045(25)(12) = 1.35 < 2.25$ ∴ Rigid Pile Action

Earthquake -

$P = 13^k$ $e = 14'$

$e/D = 14/2.5 = 5.6$ $\frac{P}{C_uD^2} = \frac{13000}{900(2.5)^2} = 2.31$ Req'd $L/D = 4.5 \pm$

$FS = \frac{25}{11.2}(1.33) = 3.0$

Broms Method (cont)

Cohesionless Case

SF : T: $\eta = \sqrt[5]{\frac{n_h}{EI}} = \sqrt[5]{\frac{60\ \#/in^3}{(3140000)(39{,}761)}} = 0.014$ $\eta L = 0.014(15)(12) = 2.52$

$2.52 > 2.0$ ∴ Flexible Pile Action

EQ: $\eta = \sqrt[5]{\frac{30\ \#/in^3}{(3140000)(39{,}761)}} = 0.012$ $0.012(25)(12) = 3.6 > 2.0$

∴ Flexible Pile Action

Resisting Moment, $M_R = f_c S$; $f_c = 1200$ psi

S - Conservatively calc. S for the transformed uncracked section (30"Ø Shaft w/ 8 #11 Verts)

$n = 10$ $A_s = 8(1.56) = 12.48\ in^2$ $A_c = \pi(15")^2 - 12.48 = 694.4\ in^2$

$A_T = 694.4 + 10(12.48) = 819.2\ in^2$

$r_T = \sqrt{819.2/\pi} = 16.1"$ $S = \pi(16.1)^3/4 = 3278\ in^3$

$M_R = 1200(3278) = 3934^{k\text{-}in} = 328^{k\text{-}ft}$

$\frac{M_B'}{D^4 \gamma K_p} = \frac{328}{(2.5)^4(.120)(3.0)} = 23.3$

From Design Charts - Ultimate Analysis of Flexible Piles
Broms 'Design of Laterally Loaded Piles,' ASCE Proc 4342 May '65

Stream Flow : Temp : $P = 10^k$ $e/D = 6.8$

$P/K_p D^3 \gamma = 3.4^{\pm}$ allow $P = 3.4(3.0)(2.5)^3(.120) = 19.1^k > 10^k$

$FS = 19.1/10 \times 1.25 = 2.4$

Earthquake: $P = 13^k < 19.1^k$ $FS = 19.1/13 \times 1.33 = 2.0$

∴ Embedment Depth of 15' below finish grade/scour o.k. for lateral load

Design of Columns Using DM-7

Case IV - Stream Flow and Temperature $1.0(D+L+S+T) \leq 1.25$

$f = 60\ \#/in^3$

EI (for gross sect.) $= (3122\ ksi)(\pi(15'')^4/4) = 1.24 \times 10^8\ k\text{-}in^2$

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

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

$P = 10^k \quad M = 170^{k\text{-}ft}$

Max. Moment (1982 DM-7, Figure 11)

@ $z/T = 0$ $\quad M_M = 1.0(170) = 170^{k\text{-}ft}$ - controls

$M_P = 0.0$

@ $z/T = 1.0$ $\quad M_M = 0.75(170) = 128^{k\text{-}ft}$

$M_P = 0.60(10^k)(73/12) = \underline{36^{k\text{-}ft}}$

$164^{k\text{-}ft}$

$M_{CASE\ IV} = 170/1.25 = 136^{k\text{-}ft}$

ext. col. $P_D = 122^k$

$P_L = (2.8^k/ft)(23'/2 + 6.65') = 51^k$

$P_{CASE\ IV} = \frac{122^k + 51^k}{1.25} = 138^k$

CASE IV (cont)

AASHTO 1.5.28, 1.5.34

$l_u = 24'$, $K = 2.1$ $r = 0.25(30") = 7.5"$ $Kl_u/r = 2.1(24)(12)/7.5 = 80.6 < 100$ ✓

$\phi = 1.0$; $\beta_d = 0.0$ (No DL Moment Transfered to Columns); $C_m = 1.0$

$EI = (E_c I_g/2.5)/(1+\beta_d) = (3122)(39761)/2.5 = 4.97 \times 10^7$ k-in^2

$P_c = \pi^2 EI/(Kl_u)^2 = \pi^2(4.97 \times 10^7)/(2.1(24)(12))^2 = 1341^k$

$\delta = \dfrac{C_m}{1 - \frac{P_u}{\phi P_c}}$ $P_u = 2.5(138) = 345$ $\delta = \dfrac{1.0}{1 - \frac{345}{1341}} = 1.35$

$M_c = \delta M = 1.35(136) = 184$ k-ft

Design Loads $M = 184/0.35 = 526$ k-ft
$P = 138/0.35 = 394^k$ $c = 1.34'$

CASE VII - Earthquake - $1.0(D+SF+EQ) \leq 1.33$

Assume Earthquake and Stream Flow don't occur simultaneously - Earthquake controls

$\gamma = 30$ #/in^3 $EI = 1.24 \times 10^8$ k-in^2 $T = \left(\frac{1.24 \times 10^8}{.03}\right)^{1/5} = 84"$ $L = 25'$

$L/T = 25(12)/84 = 3.6$

Max Moment

@ $z/T = 0$ $M_M = 1.0(182^{k\text{-}ft}) = 182^{k\text{-}ft}$
$M_P = 0$

@ $z/T = 1.2$ $M_M = 0.76(182^{k\text{-}ft}) = 138^{k\text{-}ft}$
$M_P = 0.73(13^k)(84/12) = 66^{k\text{-}ft}$ } 204 k-ft - controls

$M_{CASE\ VII} = 204/1.33 = 153^{k\text{-}ft}$

$P_{CASE\ VII} = P_D/1.33 = 122/1.33 = 92^k$

$l_u = 14'$; $k = 2.1$; $r = 7.5''$ $\quad klu/r = 2.1(14)(12)/7.5 = 47 < 100$ ✓

$$P_c = \frac{\pi^2 (4.97 \times 10^2)}{(2.1(14)(12))^2} = 3940$$

$$\delta = \frac{1.0}{1 - \frac{2.5(92)}{3940}} = 1.06$$

$M = 1.06(153) = 162^{k\text{-}ft}$

Design Loads $\quad M = 162/0.35 = 463^{k\text{-}ft}$

$P = 92/0.35 = 263^k$

$e = \frac{463}{263} = 1.76'$

∴ Case IV Controls

<u>Use 8 #11 Verts w/ 1/2" Spirals</u>

$F_y = 60{,}000$ psi

$f'_c = 3000$ psi

$D = 30''$

$D1 = 3'' + 1/2'' + 1.41''/2 = 4.20$ use $D1 = 4.25''$

#11 bar - $A_s = 1.56$ in^2 - use $N = 8$

$\left.\begin{array}{l} P = 394^k \\ M = 526^{k\text{-}ft} \end{array}\right\}$ Case IV Design Loads

AASHTO 1.5.11 (B) -

Use 1/2" Ø Spiral ties @ 6" pitch

Note: $f'_c = 4000$ 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=8.00
P=394.00
MX=526.00
MY=1.00E-8

REBAR COORDINATES
Rb=10.75 ∠=0.00
Rb=10.75 ∠=45.00
Rb=10.75 ∠=90.00
Rb=10.75 ∠=135.00
Rb=10.75 ∠=180.00
Rb=10.75 ∠=225.00
Rb=10.75 ∠=270.00
Rb=10.75 ∠=315.00

(.8)PMAX=1,410.90

ebR=12.79
MbR=6,681.90
PbR=522.25
cbR=15.24
B∠=1.09E-9

P=393.85
MX=554.92
MY=1.05E-8
cR=13.74
ecR=1.41
```

$> 526^{IK}$

O.K.

Vert. Capacity

Axial Load = $150^k + 69^k = 219^k$

Ref: Figure 4, Sergent, Hauskins & Beckwith Soil Report - See Appendix

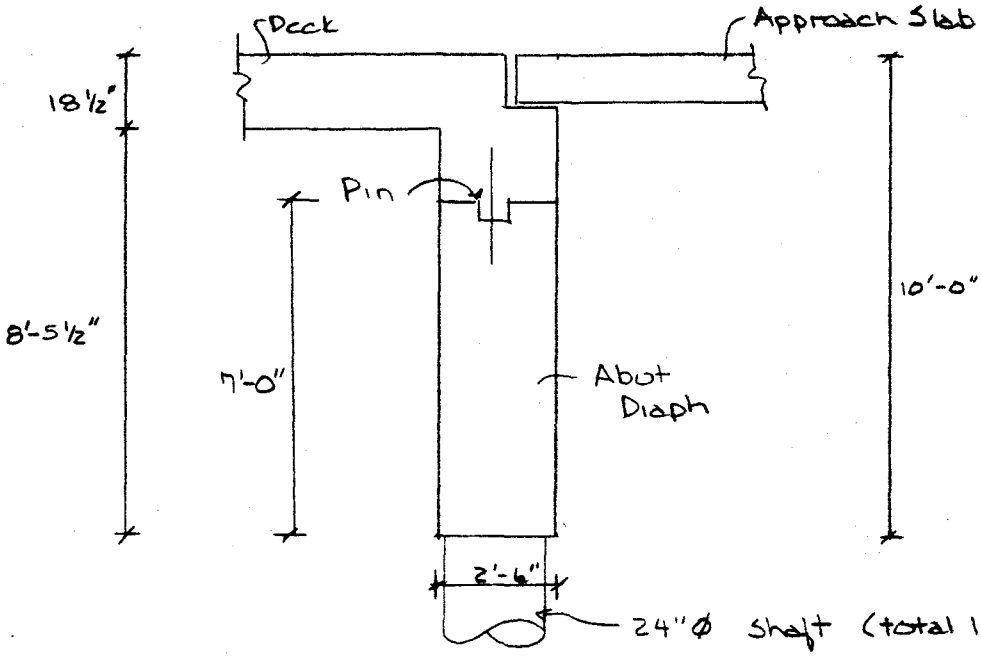
Bell Diam = 4.5' Safe Capacity @ 1299' = $275^k > 219^k$

Use 30" Ø columns w/ 8 #11 verts
& 1/2" Ø spiral ties @ 6" pitch

w/ 4.5' Ø Bell & Pier Tip Elev. = 1299'

Abutment Design

Abutment Cap



Dead Load

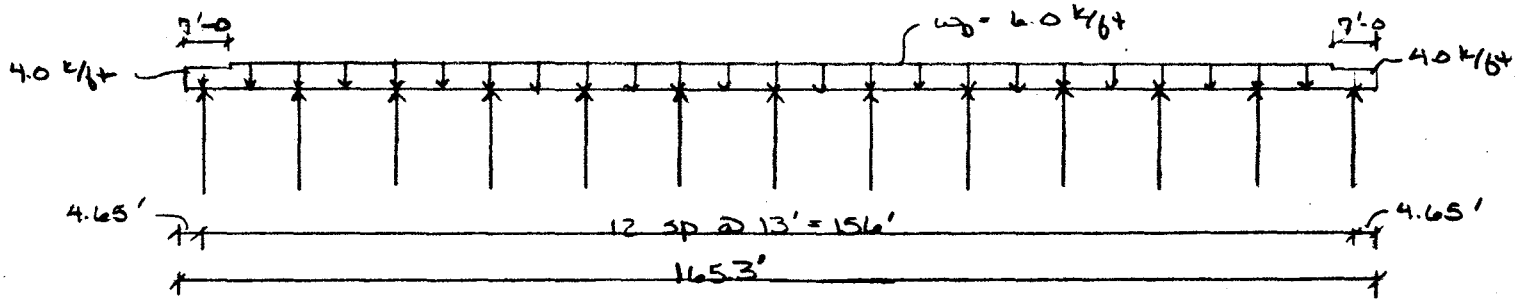
From Deck Calc: $R = 3.1\ k/ft$ Smear Barrier Load $= \left(\frac{3.1\ k/1}{.256\ ksf}\right)\left(\frac{2(0.4)+0.2}{151.29}\right) = 0.08\ k/ft$

$$w_D = \cos 58.52^\circ (3.1\ k/ft) + 0.08\ k/ft = 1.70\ k/ft$$

weight of Abutment cap $= (0.150\ k/ft^3)(2.5')(10') = 3.75\ k/ft$ use $4.0\ k/ft$

Total uniform dead load: $w_D = 1.70 + 3.75 = 5.45\ k/ft$

Use $w_D = 6.0\ k/ft$



Live Load

$76'/12' = 6.33 \Rightarrow$ use 6 traffic lanes

Ref.: Moments, Shears & Reactions for Continuous High Br.
Table 4.2 – See Appendix

$R_{LL} \approx 48^k/\text{Lane}$

$w_L \approx 6(48^k/\text{Lane})/151.29' = 1.90\ k/ft$ use $w_L = 2.0\ k/ft$

Interior caisson

$P_{DL} \approx 13'(6\ k/ft) = 78^k$
$P_{LL} \approx 13'(2\ k/ft) = 26^k$
104^k

Exterior caisson

$P_{DL} = (13'/2 + 4.65')(4\ k/ft) + (4')(2\ k/ft) = 53^k$
$P_{LL} = (4')(2\ k/ft) = 8^k$
61^k

D+L:

$\text{Max} \pm M \approx 0.1\, w\ell^2 = 0.1(8\ k/ft)(13')^2 = 135\ k\text{-}ft$

$d = 7'\text{-}0'' - 6'' - 2(0.75'') - 0.875''/2 = 76.0''$ use $d = 75''$

Try 3 #7 T&B $A_s = 3(0.60) = 1.80\ in^2$ $n = 29{,}000/57\sqrt{3500} = 8.6$
use $n = 9$

$n\frac{A_s}{bd} = 9\frac{1.80}{(30)(75)} = 0.0072$ $k = 0.113$ $j = 0.962$

$f_s = \frac{M}{A_s jd} = \frac{135(12)}{(1.80)(.962)(75)} = 12.5\ ksi$

$f_c = \frac{kf_s}{n(1-k)} = \frac{.113(12.5)}{9(1-.113)} = 0.18\ ksi$ ∴ O.K. use 3 #7 T&B

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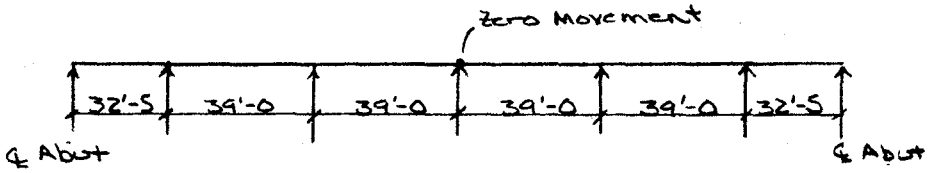
Lateral Load Resnt

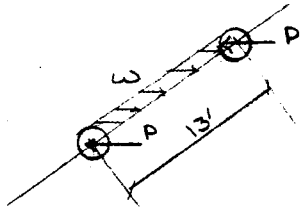
- Assume abutment cap moves as a rigid body with slab

Cap Movement due to temp:



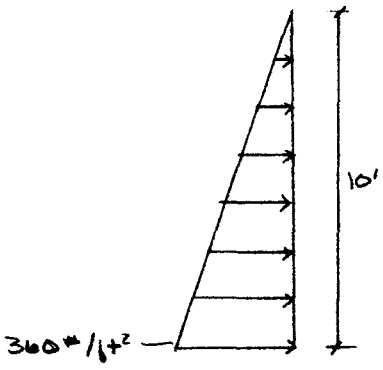
$\delta = (.0000006)(40°)(110.4')(12) = 0.32''$

P_δ /caisson = 31^k – See Caisson Design Calcs



$w = 31^k/13' = 2.4\ ^k/LF$

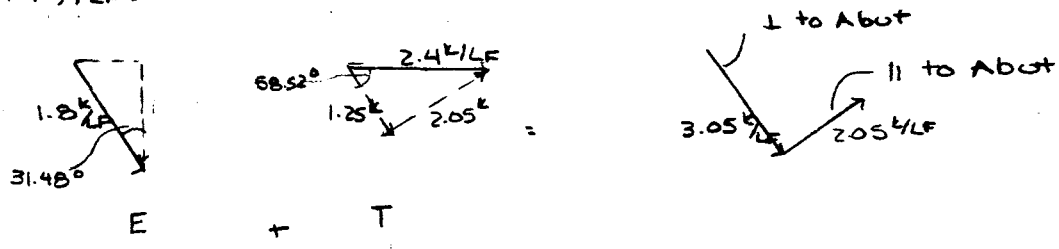
Earth Pressure



Active Earth Pressure = 36 pcf

Long Force = 1/2 (360)(10) = 1.8^k/L.F.

(E+T)/LF:

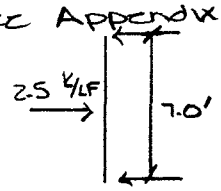


E+T Long. Force = 3.05 k/LF

Design Load: $3.05/1.25 = 2.44$ k/LF (AASHTO 1.2.22 - Case IV)

use 2.5 k/LF Ref: 'Bridge Planning & Design Manual, Vol IV', pg 6-22
See Appendix

use #5 @ 12" $M \approx 2.5(7')/4$
$= 4.4$ k-ft



$d = 30'' - 3'' - .625/2 = 26.7''$ use $d = 26''$ $A_s = 0.31$ in^2

$n \frac{A_s}{bd} = 8 \frac{0.31}{(12)(26)} = 0.0079$ $k = 0.118$ $j = 0.961$

$f_s = \frac{4.4(12)}{(0.31)(.961)(26)} = 6.81$ ksi $\therefore$ O.K use #5 @ 12"

$v_{max} = 2.5/(12)(26) = 0.008$ ksi $< \frac{1}{2}(0.95\sqrt{3500}) = 28$ psi

AASHTO 1.5.7 (D)

$\rho_{min} = 0.002$

$A_g = (12')(26'') = 312 \text{ in}^2$ $\quad A_s = 2(0.31) = 0.62 \text{ in}^2$

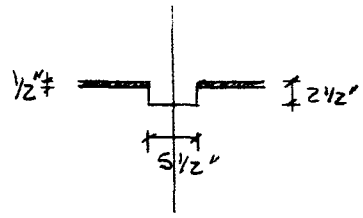
$\rho = \frac{0.62}{312} = 0.002 = 0.002$ o.k.

AASHTO 1.4.7 (B) Temp. Reinf. Req'd = 1/8 in²/ft ht.

<u>Use #5 bars @ 12"</u> $A_s = 0.31 \text{ in}^2/\text{ft} > 0.125 \text{ in}^2/\text{ft}$

<u>Shear Key (pin)</u>

(AASHTO 1.5.29 (D))



Req'd $A_{vf} = V / f_s \mu$

$\mu = 1.4$ $\quad f_s = 20 \text{ ksi} \times 1.25$

$V = .672 \text{ k/LF} + 1.25 \text{ k/LF} = 1.92 \text{ k/LF}$

Req'd $A_{vf} = 1.92 \text{ k/LF} / (20 \text{ ksi})(1.25)(1.4) = 0.055 \text{ in}^2/\text{LF}$

Use #4 @ 12" $\quad A_{vf} = 0.20 \text{ in}^2/\text{LF}$

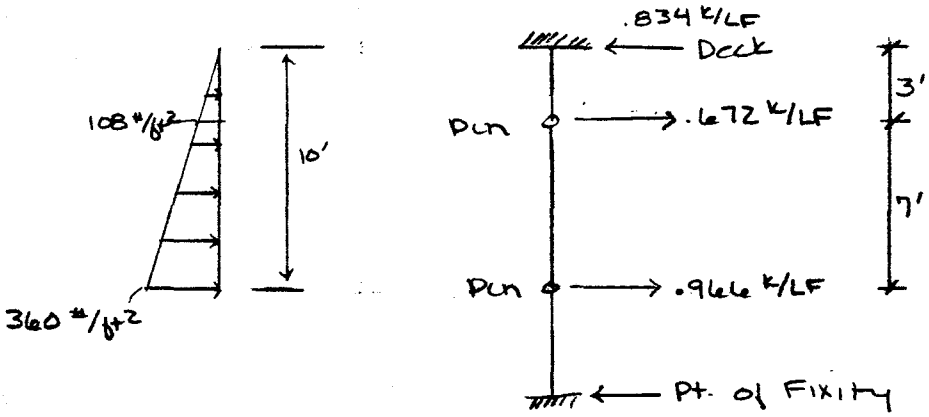
Note: Use $f'_c = 4000$ psi Redesign Not Req'd

Caisson Design

Assumptions:

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

Lateral Load due to Earth Pressure



Interior Caissons Support 13'

Lateral Load / int. shaft = (.966 k/LF)(13') = 12.6 k

Exterior Caissons Support 13'/2 + 4.65' = 11.15'

Lateral Load / ext. shaft = (.966 k/LF)(11.15') = 10.8 k

Ref: NAVDOCKS DM-7:

$f = 0.06\ ^k/in^3$ 24" Ø Caisson $I = \pi(12)^4/4 = 16{,}286\ in^4$

$E = 3122$ ksi $EI = 5.08 \times 10^7$ k-in²

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{3122(16286)}{.06}\right)^{1/5} = 61.1''$$

L: 1) 1340.74 - 10 - 1299 = 32'

2) 1339.02 - 10 - 1299 = 30' use L = 31'

$$L/T = 31(12)/61.1 = 6.1$$

Temperature Forces:

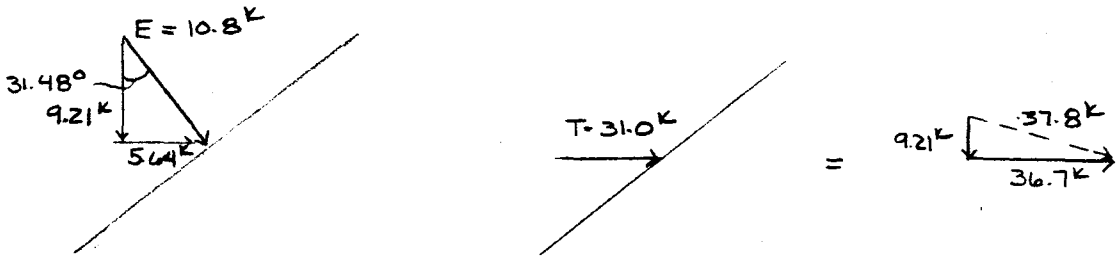
$$\delta = 0.32'' \quad F_\delta = 2.3 \quad P_\delta = \frac{\delta EI}{F_\delta T^3} = \frac{0.32(5.08 \times 10^7)}{2.3\,(61.1)^3} = 31.0^k/\text{shaft}$$

Axial Loads: Exterior : $P_{D+L} = 61^k$

Interior : $P_{D+L} = 104^k$

∴ Load Case IV on exterior shafts controls design

E+T:



E+T - Lateral Load = 37.8^k

M_{max} @ $z/T = 1.3$ $M = 0.77(37.8^k)(61.1/12) = 148$ k-ft

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Case IV : $M = 148/1.25 = 119^{k\text{-}ft}$

$P = 61^k/1.25 = 48.8^k$

- Slenderness Need Not Be Considered - Caisson fully embedded

$M_u = 119/0.35 = 340^{k\text{-}ft}$

$P_u = 48.8/0.35 = 139^k$

Use $f'_c = 4000$ psi

Use 8 #11 Verts

& 1/2" Ø Spiral @ 6" Pitch

Note: $f'_c = 4000$ psi req'd

- Use typically @ substructure

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

                    "P"
FY=60,000.00
Fc=4,000.00
D=24.00
D1=4.25
AS1=1.56
N=8.00
P=139.00
MX=340.00
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=1,256.92

ebR=11.56
MbR=4,802.55
PbR=415.44
cbR=11.69
B∡=1.69E-9

P=138.17
MX=360.80
MY=1.06E-8
cR=9.03
ecR=2.61
```

> 340^{lk}

o.k.

Bearing Capacity of Caisson

@ 1299' Soil Bearing Capacity = 185^k per SHB report

Axial Load = 104^k < 185^k - See Appendix

∴ O.K. Tip Elev. = 1299'

Investigate Need For Additional Caisson due to Skew:

Ref: Bridge Planning & Design Manual

No. of Piles Required under 1/4 Support @ Obtuse corner:

$$\frac{\text{"k" x reaction on piles}}{\text{allow. pile value}}$$

Use "k" = 0.50

Reaction on piles: DL: (6.0 ^{k}/ft)(151.3') + 4.0 ^{k}/ft (14') = 964^k
LL: (48 ^{k}/lane)(6 lanes) = 288^k
1252^k

Allow. Pile Value: 185^k (Soil Bearing Capacity Controls)

Reqd No. of Piles = $\frac{0.50(1252)}{185}$ = 3.38

∴ Shafts @ 13'-0" o.c. O.K.

use 24" Ø Shafts w/ 8 #11 Verts ;

1/2" Ø Spiral ties @ 6" pitch

; tip elev = 1299' (total 13 per abut.)

QUANTITIES

QUANTITIES

CONCRETE

SUPER-STRUCTURE $f'_c = 4500$

DECK $220'(18.5/12)(79')(1/27) =$ 992.4 cy

HAUNCH GIRDER $\{5.25'(7.5/12)(79/\cos 58.5^\circ) - 2(1/2)(7.5/12)(7.5/12)(79/\cos 58.5) + (10''/12)(10''/12)(3''/12)(7)\}(5)(1/27) =$ 81.2 cy

DECK TOES: $\{2.5'(1.21')(79/\cos 58.5^\circ) + (8''/12)(6.5''/12)(79/\cos 58.5) + (2.5/12)(5.5''/12)(79/\cos 58.5)\}(2)(1/27) =$ 39.0 cy

1112.6 cy

$f'_c = 4000$

ABUTMENTS: $\{(7')(2.5')(165.28') + (165.28 - 79/\cos 58.5)(2.5')(2.5')\}(2)(1/27) =$ 220.8 cy

COLUMNS: $3.14(2.5')(2.64')(7 \text{ cols/pier})(5 \text{ piers})(1/27) =$ 26.9 cy

247.7 cy

$f'_c = 3000$

APPROACH SLABS: $15'\cos 58.5^\circ(68'/\cos 58.5^\circ)(1')(2)(1/27) =$ 75.6 cy

BARRIERS: $2(220')(2.5 \text{ ft}^2)(1/27) =$ 40.7 cy

CURB: $(8''/12'')(1')(220)(1/27) =$ 5.4 cy

121.7 cy

DRILLED SHAFTS

30" Φ SHAFT W/4.5 BELL $7(5)(1335-1299) =$ 1260 FT

24" Φ SHAFT $13(2)(31') =$ 806 FT

30" Φ

CONC: $\{36'(3.14)(1.25)^2 + (3.14)(4.5)(2.5^2-1.25^2) + 3.14(1)(2.25^2-1.25^2)\}1/27 =$ 7.22 cy/SHAFT

STEEL $36(8)(5.313) + 73(2)(3.14)(0.667) =$ 1836.07 #/SHAFT

24" Φ

CONC. $31'(3.14)(1)^2/27 =$ 3.61 cy/SHAFT

STEEL. $31'(8)(5.313) =$ 1581.7 #/SHAFT

EXCAVATION

EXCAVATION $2(165.28' + 3')(2.5+3)(4')(1/27) =$ 274.2 cy

BACKFILL $= 274.2 \text{ cy} - 4(2.5)(165.28)(1/27)(2) + 2(5')(170')((2+15)/\cos 58.5)(1/27) + 2(1/2)(5)(2.5)(170)(1/27) =$ 789.8 cy

---

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Consulting Engineers

project POWER RD @ FLDWAY
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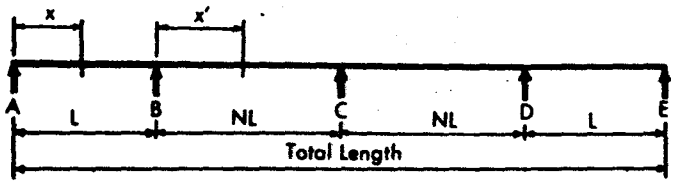
checked by
date 10-5-84 sh. 43 of

Appendix

TABLE 4.2

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



N=1.2

Total Length Ft.	Span Length "L"	Max. Reaction Kips.			Max. Shear Kips.			Max. Moment Kip.-ft.				Impact					Dist.-Ft.	
		at A	at B	at C	in AB at B	in BC at B	in BC at C	in AB at X	at B	in BC at X'	at C	I	II	III	IV	V	X	X'
120	27.3	44.5	61.8	61.9	-50.2	51.7	-51.6	200.8	-182.7	200.6	-187.1	.300	.300	.300	.270	.263	9.7	17.9
140	31.8	47.7	64.2	64.3	-54.1	54.9	-54.8	253.8	-216.6	256.3	-221.6	.300	.300	.300	.256	.248	11.5	20.6
160	36.4	50.3	65.9	65.9	-56.9	57.4	-57.3	308.4	-251.1	314.7	-261.5	.300	.300	.296	.244	.236	13.9	23.4
180	40.9	52.5	67.1	67.1	-59.1	59.3	-59.2	372.8	-299.6	374.9	-312.2	.300	.294	.287	.233	.224	16.6	26.1
200	45.5	54.3	68.0	67.9	-60.7	60.8	-60.7	437.6	-351.8	436.5	-367.0	.293	.286	.278	.222	.214	18.4	28.8
220	50.0	55.8	68.6	69.5	-62.0	62.0	-61.9	503.0	-407.8	498.9	-425.7	.286	.278	.270	.213	.204	20.3	31.5
240	54.5	57.1	72.3	73.4	-63.0	63.0	-62.9	568.9	-467.6	562.2	-488.5	.278	.270	.263	.204	.195	22.3	34.3
260	59.1	58.2	76.2	77.4	-63.9	63.9	-63.8	635.1	-531.2	625.9	-555.2	.272	.263	.255	.196	.187	24.2	37.0
280	63.6	59.1	80.0	81.3	-64.6	64.6	-64.5	701.6	-598.6	690.1	-626.0	.265	.256	.248	.189	.180	26.2	39.7
300	68.2	59.9	83.9	85.3	-65.2	65.1	-65.1	768.3	-669.8	754.6	-700.8	.259	.250	.242	.182	.173	28.0	42.5
320	72.7	60.7	87.8	89.2	-65.7	65.7	-65.6	835.2	-744.8	819.4	-779.6	.253	.244	.236	.175	.167	30.0	45.2
340	77.3	61.3	91.6	93.2	-66.2	66.1	-66.0	902.3	-823.5	884.5	-862.4	.247	.238	.230	.169	.161	31.9	48.0
360	81.8	61.9	95.5	97.1	-66.6	66.5	-66.4	969.4	-906.1	949.8	-949.2	.242	.233	.224	.164	.156	33.9	50.8
380	86.4	62.4	99.3	101.1	-66.9	66.8	-66.8	1036.7	-992.4	1015.2	-1040.0	.237	.227	.219	.159	.150	35.9	53.5
400	90.9	62.9	103.2	105.0	-67.2	67.1	-67.1	1104.0	-1082.5	1080.7	-1134.8	.232	.222	.214	.154	.146	37.9	56.2
420	95.5	63.3	107.1	109.0	-67.5	67.8	-67.5	1171.5	-1176.5	1146.4	-1233.7	.227	.217	.209	.149	.141	39.8	59.0
440	100.0	63.7	110.9	112.9	-67.7	69.8	-69.5	1238.9	-1274.2	1212.2	-1336.5	.222	.213	.204	.145	.137	41.8	61.6
460	104.5	64.1	114.8	116.9	-69.0	71.7	-71.4	1306.5	-1375.7	1278.1	-1443.4	.218	.208	.200	.141	.133	43.7	64.4
480	109.1	64.4	118.6	120.8	-70.9	73.7	-73.4	1374.1	-1481.0	1344.0	-1554.2	.214	.204	.195	.137	.129	45.6	67.1
Impact		I	IV	V	I	III	III	I	II	III	III							
Dead Load		.3774 x wL	1.2259 x wL	1.1934 x wL	-.6226 x wL	.6033 x wL	-.5967 x wL	.0712 x wL^2	-.1226 x wL^2	.0593 x wL^2	-.1187 x wL^2						.3774 x L	.6033 x L

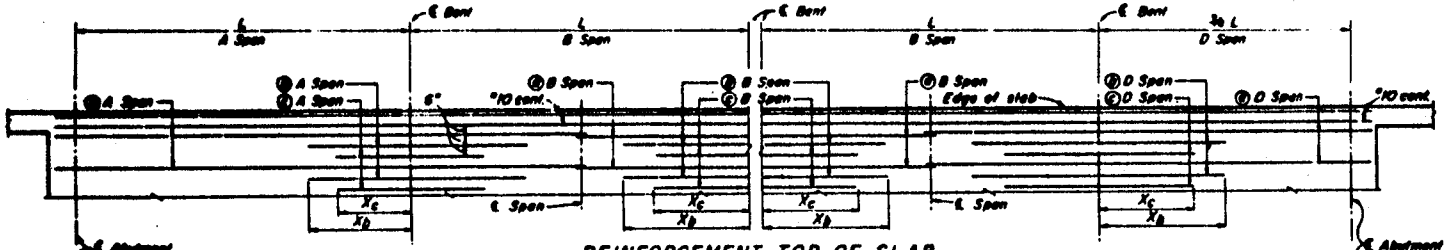
From: 'Moments, Shears and Reactions For Continuous Highway Bridges' AISC 1966

Power Rd @ Floodway 83-058

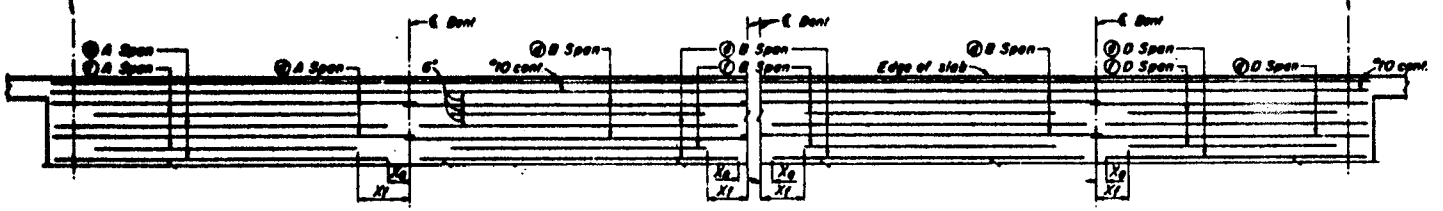
Length of Span	16'			18'			20'			22'			24'			26'			28'			30'			32'			34'			36'			38'			40'			42'			44'		
REINFORCEMENT — Top of Slab — Bottom of Slab																																													
Top of Slab ① Bars	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9
Top of Slab ① Length	17'-0"	16'-0"	22'-0"	20'-0"	20'-0"	25'-0"	23'-0"	22'-0"	27'-0"	26'-0"	24'-0"	30'-0"	29'-0"	27'-0"	32'-0"	32'-0"	29'-0"	35'-0"	35'-0"	31'-0"	37'-0"	38'-0"	33'-0"	40'-0"	38'-0"	37'-0"	42'-0"	38'-0"	39'-0"	45'-0"	40'-0"	41'-0"	48'-0"	42'-0"	44'-0"	50'-0"	45'-0"	45'-0"	53'-0"	47'-0"	48'-0"	55'-0"	50'-0"	49'-0"	58'-0"
Top of Slab ② Bars	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9
Top of Slab ② Length	12'-0"	10'-0"	14'-0"	13'-0"	12'-0"	16'-0"	15'-0"	14'-0"	18'-0"	16'-0"	15'-0"	19'-0"	18'-0"	16'-0"	20'-0"	17'-0"	16'-0"	21'-0"	19'-0"	18'-0"	22'-0"	20'-0"	19'-0"	23'-0"	21'-0"	20'-0"	24'-0"	22'-0"	21'-0"	25'-0"	23'-0"	22'-0"	26'-0"	24'-0"	23'-0"	27'-0"	25'-0"	24'-0"	28'-0"	26'-0"	25'-0"	29'-0"	27'-0"	26'-0"	30'-0"
Bottom of Slab ① Bars	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9
Bottom of Slab ① Length	9'-0"	8'-0"	10'-0"	10'-0"	9'-0"	11'-0"	11'-0"	10'-0"	12'-0"	12'-0"	11'-0"	13'-0"	13'-0"	12'-0"	14'-0"	14'-0"	13'-0"	15'-0"	15'-0"	14'-0"	16'-0"	16'-0"	15'-0"	17'-0"	17'-0"	16'-0"	18'-0"	18'-0"	17'-0"	19'-0"	19'-0"	18'-0"	20'-0"	20'-0"	19'-0"	21'-0"	21'-0"	20'-0"	22'-0"	22'-0"	21'-0"	23'-0"	23'-0"	22'-0"	24'-0"
Bottom of Slab ② Bars	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9	#7	#8	#9
Bottom of Slab ② Length	6'-0"	5'-0"	7'-0"	6'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"	7'-0"	6'-0"	8'-0"
t = Thickness of slab	10½"			11½"			12"			12½"			13½"			14"			15"			16"			16½"			17"			17½"			18"			19"			19½"			20"		
Approx. Concrete per Span (C.Y.)	Less Fill	18.8	14.1	11.4	18.2	17.3	15.1	21.0	20.0	16.0	24.0	23.0	18.2	29.1	27.0	21.4	31.5	30.3	23.9	36.7	35.0	27.2	41.3	40.0	31.4	46.4	44.0	33.0	49.3	48.1	37.5	54.0	52.5	40.4	58.3	57.0	44.3	64.9	63.3	49.0	69.8	68.2	52.8	75.0	73.3	56.*
	Sheet-Rock	133	118	99	139	130	100	188	183	118	200	125	138	240	210	160	290	220	155	310	250	175	340	280	190	370	310	210	400	340	220	430	370	250	480	410	260	550	430	290	590	480	340	635	530	340
Deflection at Midspan (ft)	01'	00'	00'	01'	00'	00'	00'	00	00'	03'	01'	00'	03'	01'	00'	04'	01'	00'	05'	01'	00'	06'	01'	00'	07'	02'	00'	08'	02'	00'	09'	03'	02'	11'	03'	02'	12'	04'	03'	14'	04'	03'	16'	05'	04'

① Add ½" for "Severe Environment" & adjust concrete quantity.

* Length includes allowance for one additional splice in 'C' or 'D' Span



REINFORCEMENT-TOP OF SLAB



REINFORCENT-BOTTOM OF SLAB

NOTES:

Top reinforcement for 'B Span' to be map'ed when adjacent to a hinge. See "Slab Details" sheet.

'D' Span lengths are actually 3½ c' ½" longer than shown in the heading (Heading 24' 'A & B' Spans are actually 24', 'D' Spans is 18').

LIVE LOADING: HS 20-44 & ALT

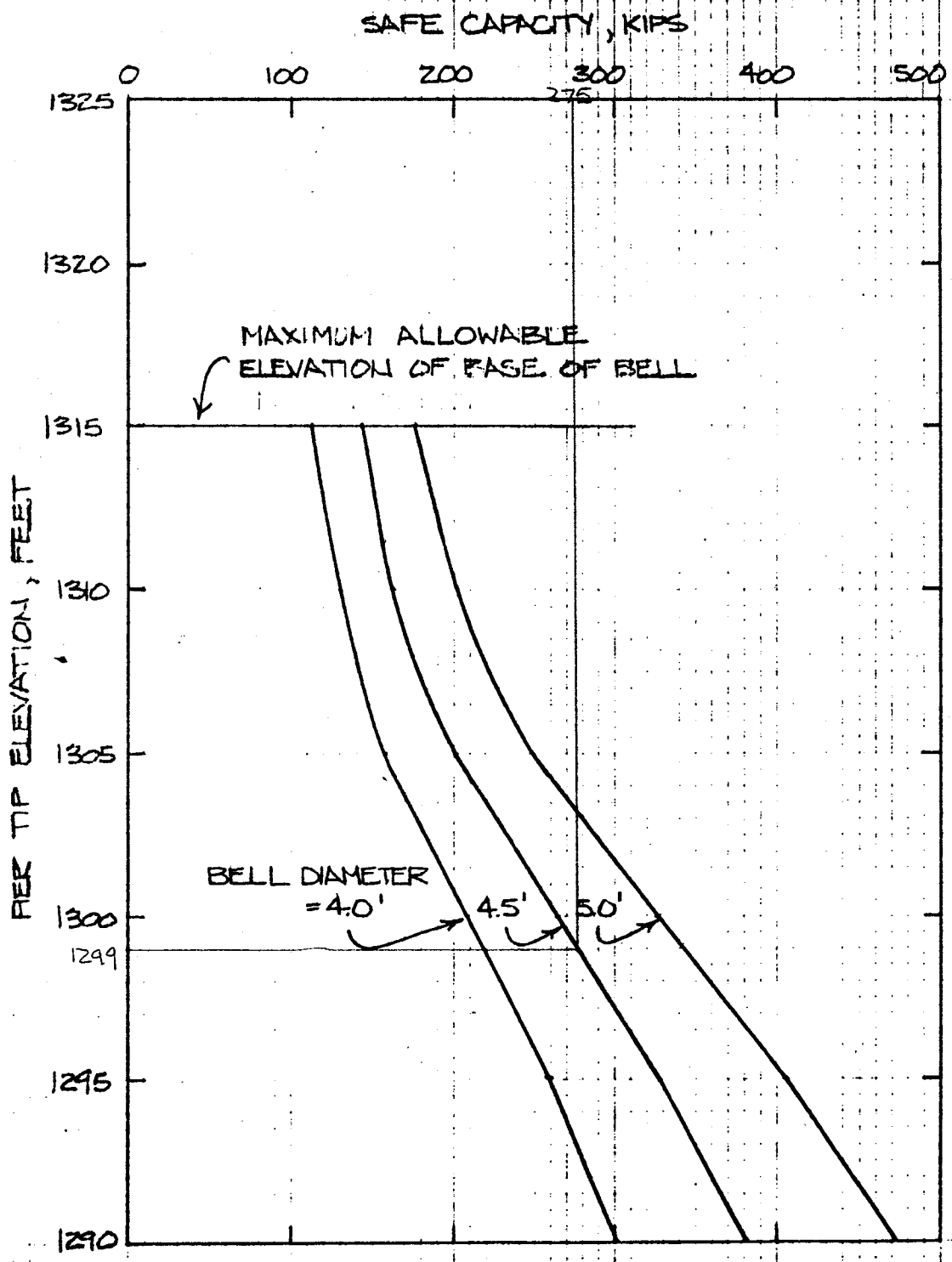
STANDARD SLAB BRIDGE

SLAB DETAILS-MULTI SPAN

Power Rd & Floodway 83-58

FIGURE 4

SAFE DOWNWARD CAPACITIES FOR BELLED PIERS.



Power Rd @ Highway 83-58

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

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

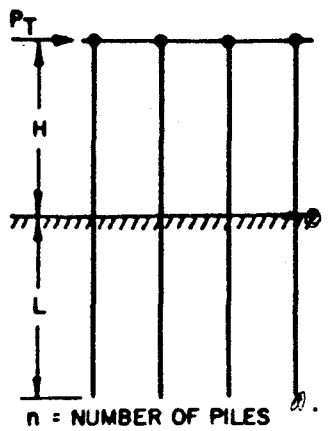
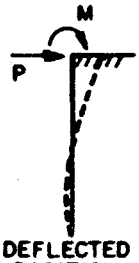
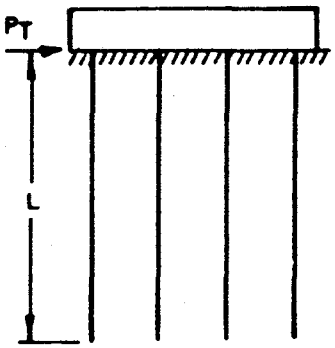
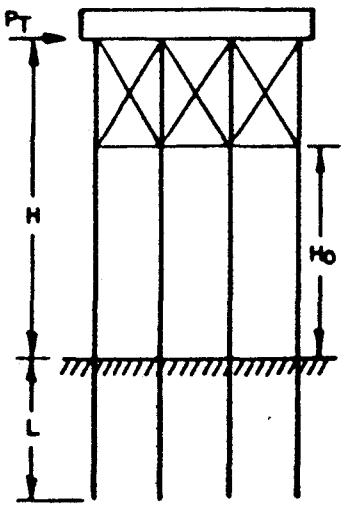
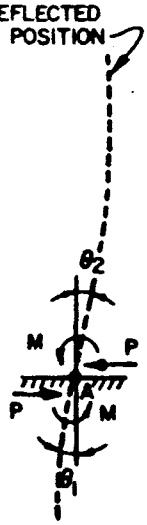
CASE I. FLEXIBLE CAP, ELEVATED POSITION		
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$ 	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. 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: $\theta_1 = F_\theta (\frac{PT^2}{EI}) + F_\theta (\frac{MT}{EI})$ 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

Power Rd @ Floodway
83-58

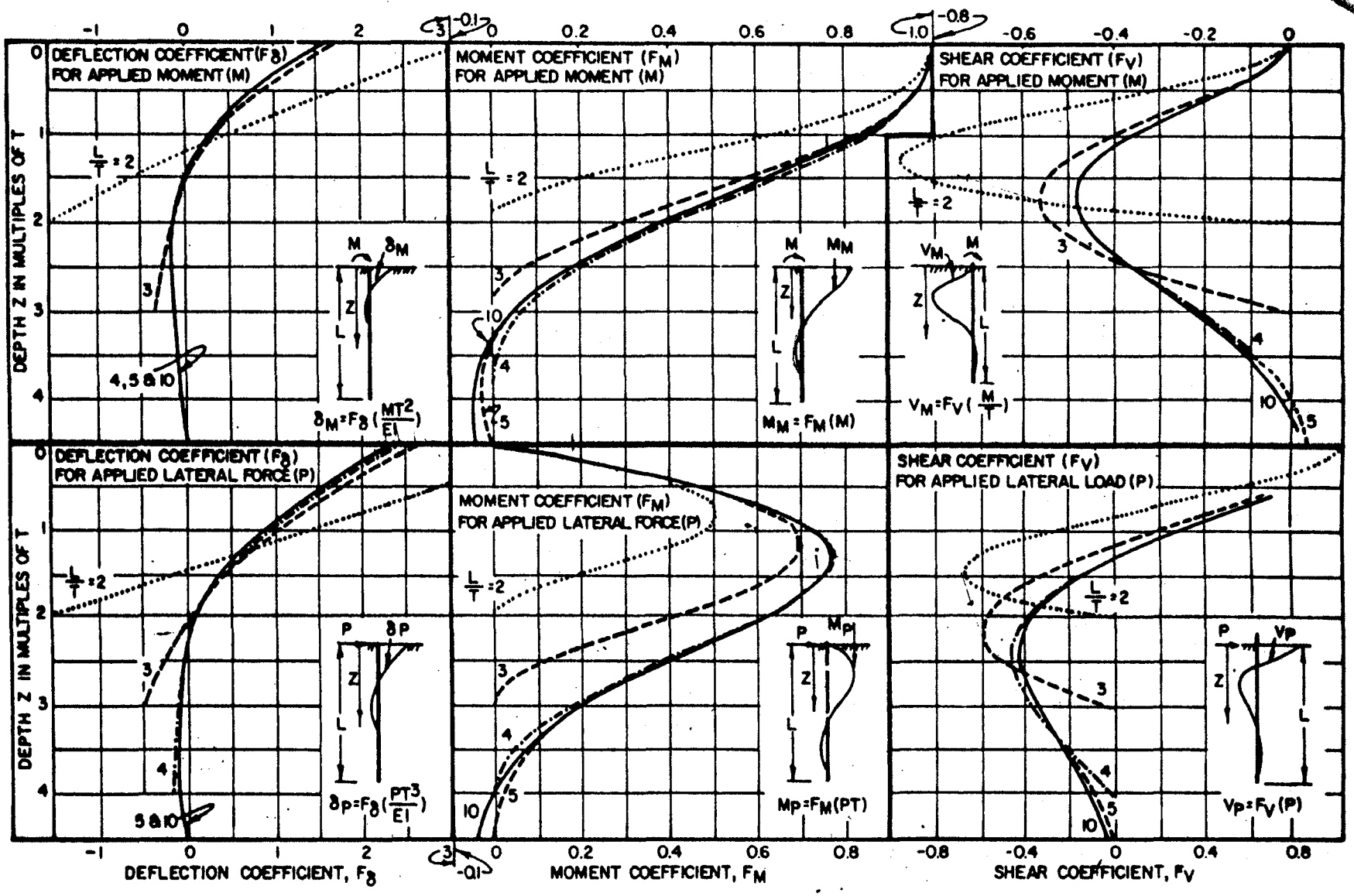


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

Power Rd & Flagpole
83-58

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	$EI = 3\times10^7 \times 361.5$
	$k_h = 80$ pci (stiff clay)	$n_h = 10$ pci
	$\beta = \sqrt[4]{\frac{80\times12}{4\times108.45\times10^8}} = 0.0120$	$\eta = \sqrt[5]{\frac{10}{3\times10^7\times361.5}} = 0.0156$
	$\beta L = 0.0120\times240 = 2.88 > 2.25$ Flexible Action	$\eta L = 0.0156\times240 = 3.74 > 2.0$ Flexible Action

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

II ULTIMATE ANALYSIS OF FLEXIBLE PILES

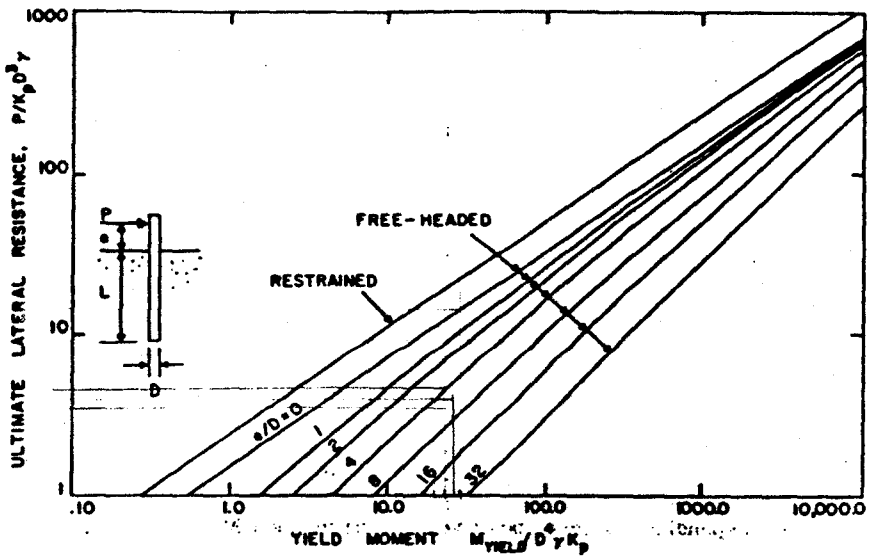
A. Analysis consists of computing the maximum bending moment producing failure in the pile section. Then the ultimate lateral load which produces the failure moment corresponds to the ultimate capacity of the pile. The margin of safety for a particular load is the ratio of the bending moment produced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

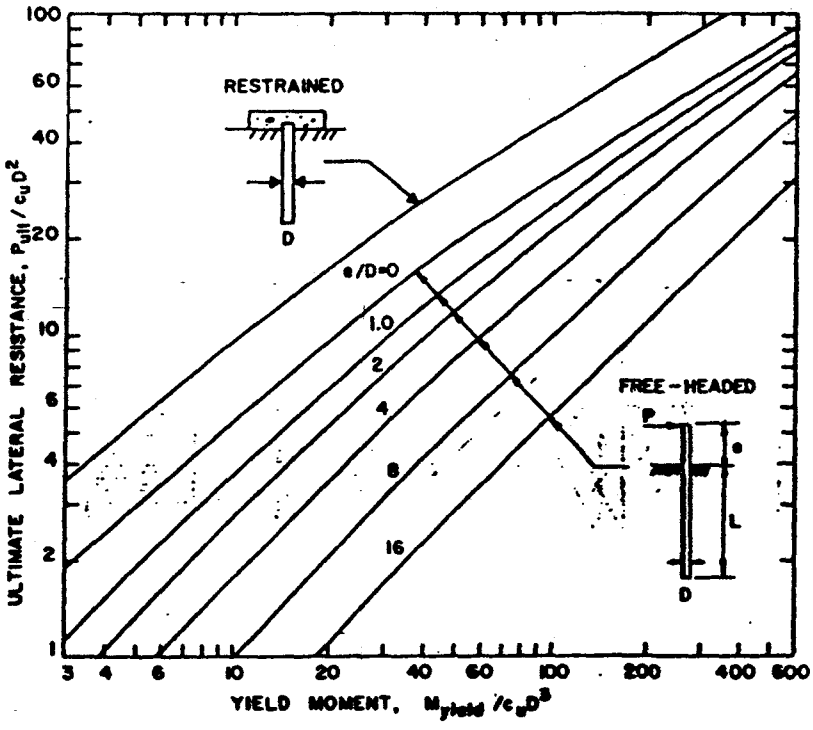
$M_{resisting} = \frac{fI}{c} = \frac{20^{*} \times 31.5}{6_{in} \times 12^{in}/ft}$
= 100 ft-kips

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

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

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

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

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

Brams method

Power Rd @ Floodway
83-58 F.

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

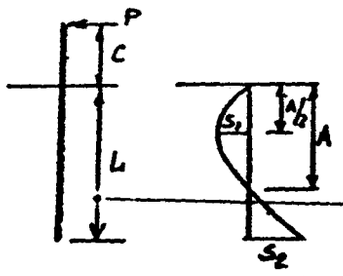
Type of Soil →	Clay	Sand
"Rigidity Factor"	$\beta = \sqrt[4]{\frac{k_h D}{4EI}}$ $\frac{\#}{in^3}$ in / $\frac{\#}{in^2} \times in^4$: $\sqrt[4]{\frac{1}{in^4}}$ (PILE UNITS)	$\eta = \sqrt[5]{\frac{n_h}{EI}}$ – PILE DEPARTURE
"Rigid action" if	$\beta L < 2.25$ (Ref 3)	$\eta L < 2.0$ (Ref 4)
Example	$k_h = 80$ 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"



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

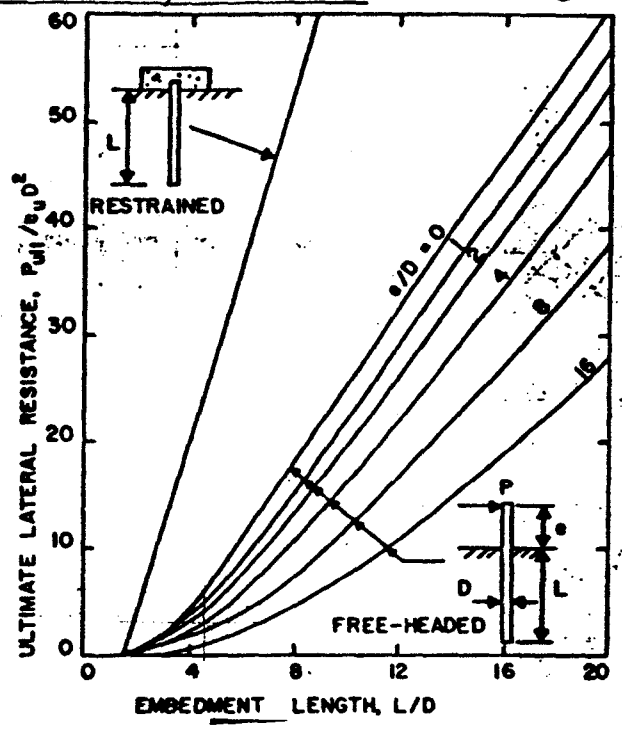
$$S_1 = \frac{1.5P(3L + 4C)^2}{L^2(4L + 6C)}$$ (usually governs design)

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

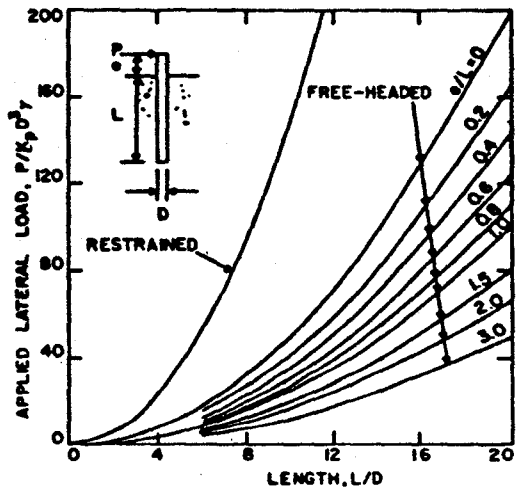
Additional equations for pile design available in Ref

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

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

$e = 0$ $D = 2'$

$P = 30$ kips

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

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

Example:

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

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

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

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

$\frac{L}{D} = 14$ $L = 14'$ (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 DON'T 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.

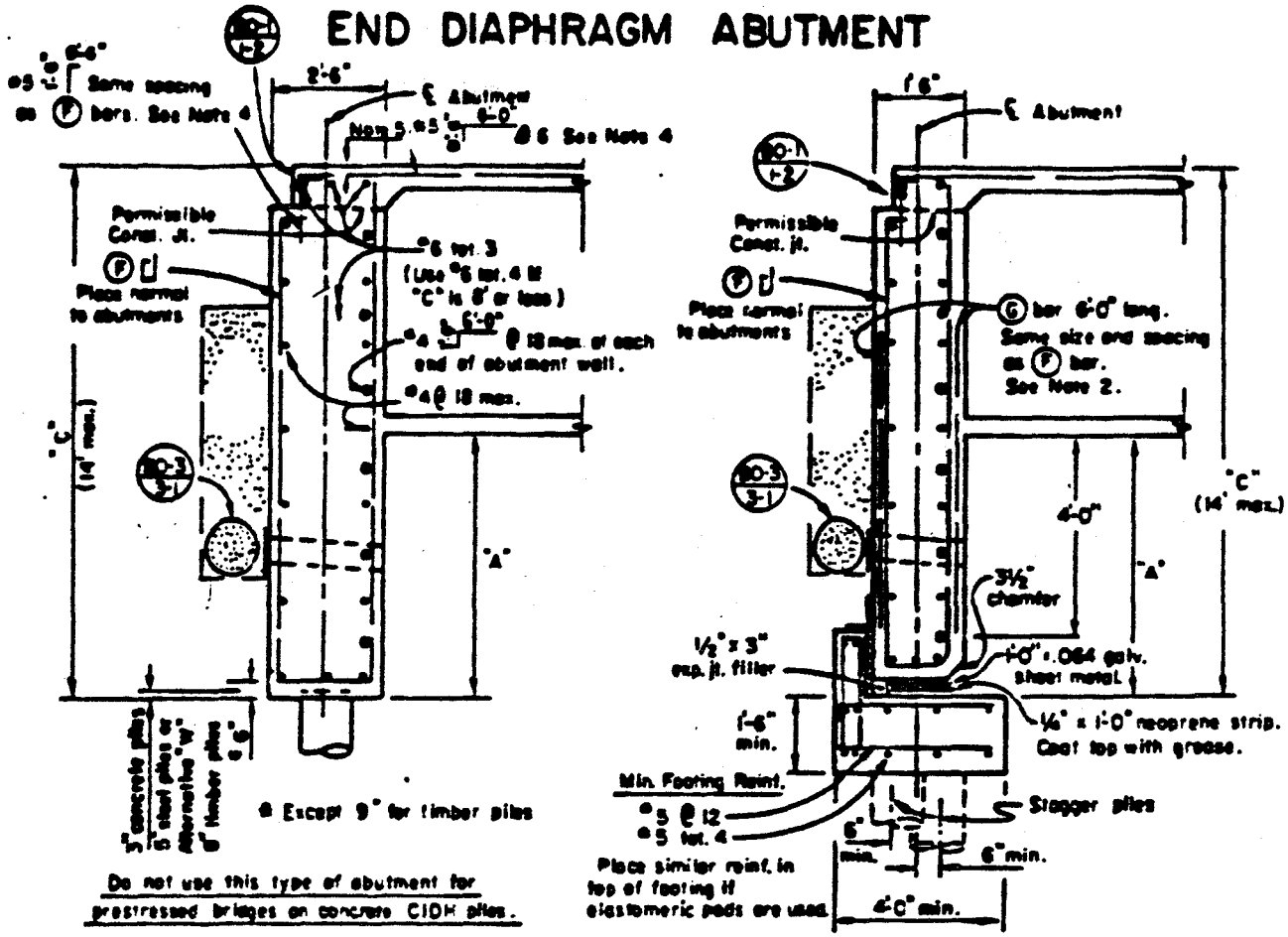
IV ELASTIC ANALYSIS OF RIGID PILES

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

END DIAPHRAGM ABUTMENT



(F) [U] BARS, #3 [U] (F) 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

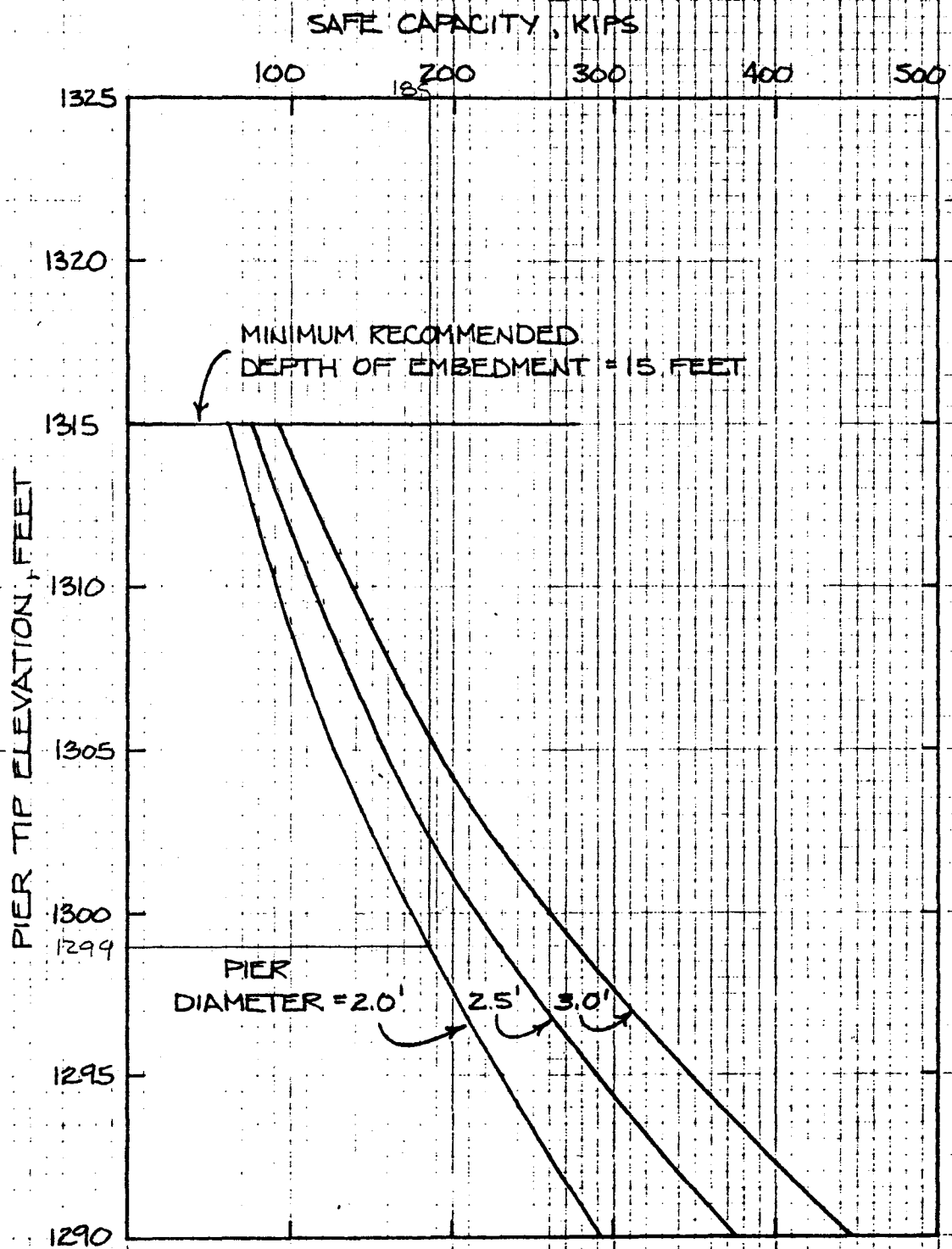
NOTES

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.

Power Rd @ Floodway
83-58

FIGURE 1

SAFE DOWNWARD CAPACITIES
FOR STRAIGHT PIERS.
- ABUTMENT PIERS -



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83-58

Project: GUADALUPE + POWER RD. BRIDGES
Job No: EB4-2
Computed by: KF Ckd. by:
Date: 2/13/84 Page ___ of ___