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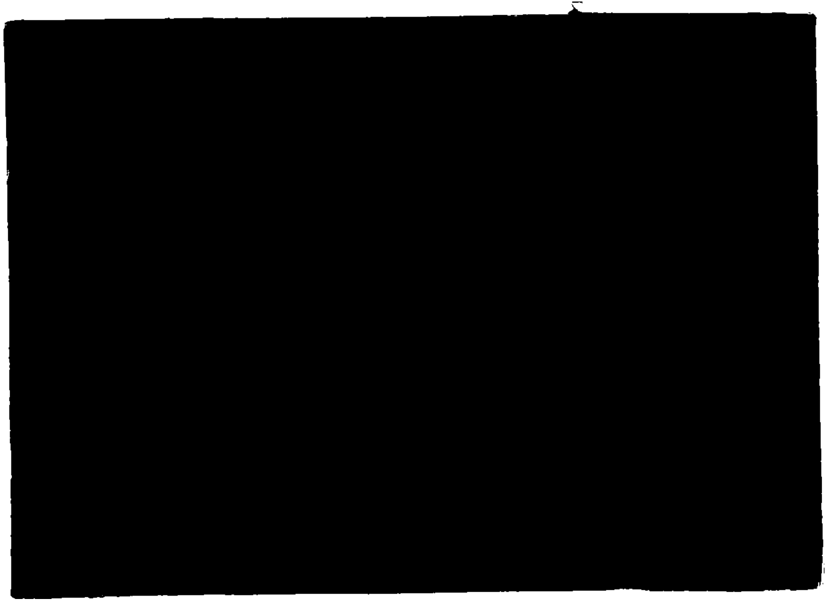

GUIDEWAY STRUCTURAL DESIGN AND POWER/PROPULSION/BRAKING IN RELATION TO GUIDEWAYS

APPENDIX B

FINAL REPORT
JANUARY 1993

GUIDEWAY STRUCTURAL DESIGN AND POWER/PROPULSION/BRAKING IN RELATION TO GUIDEWAYS

FINAL REPORT

APPENDIX B

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MAGLEV GUIDEWAY STRUCTURAL DESIGN

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MAGLEV GUIDEWAY STRUCTURAL DESIGN

1. INTRODUCTION

1.1 Maglev

Magnetically levitated high speed ground transportation, or "maglev", represents the latest evolution in advanced transportation technology. Vehicles glide above their guideway, supported, guided, and propelled by magnetic forces, at speeds that can exceed 300 miles per hour. Such systems can offer an attractive transportation alternative for many trips of 100 miles to 600 miles in length, reducing air and highway congestion, and making room for more efficient long-haul air service at crowded airports. Maglev may become the technology of choice for high-speed surface transportation in the 21st century.

1.2 Maglev Guideway Girder Design

The maglev guideway presents a new challenge in structural design. Two major areas of concern that differ maglev from the traditional transit system in the guideway girder design using reinforced concrete are:

1. **magnetic field interference** - In order to limit the potential interaction between the superconducting levitation and propulsion systems and any metal reinforcing in the girder, a minimum space of one meter is required between the superconducting systems and the reinforcing material. Metal reinforcing, especially that in the upper half of the girder section, heavily reduces the maglev system performance.
2. **amount of allowable deflection** - Deflection will affect maglev vehicle performance and ride quality. Depending on either EMS (electromagnetic system) or EDS (electrodynamic system) is used, the maglev vehicles glide only about 2 or 4 in. above the guideway. It has been suggested that the deflection due to live load plus impact should not exceed more than 1/2000 of the span.

To deal with the first concern, two kinds of girders are considered in the guideway design:

1. simply supported reinforced concrete girder
2. simply supported prestressed concrete girder with constant tendon eccentricity

They are designed in such a way that there is no major (longitudinal) compressive reinforcement at the upper half of the section and the major (longitudinal) tensile reinforcement is placed at the lower half of the section. The reinforced concrete girder uses the steel rebars as the major reinforcement while the prestressed concrete girder uses standard seven-wire steel strands. Again, since no metal reinforcing is allowed within one meter from top of the girder, non-magnetic fiberglass-reinforced plastic (FRP) rebars are used as the non-major (shear and slab) reinforcement. Although FRP rebar has twice the tensile strength of steel at one-fourth of the weight, it has very low modulus of elasticity (20 to 25 percent that of steel) and does not exhibit yielding or ductility. It can not be considered as the major reinforcing material in the girder without extensive research.

As to the second concern, the deflection requirement must be incorporated in the girder design.

1.3 Maglev Guideway Girder Sections

Five typical guideway configurations are considered in this study. They are:

Type I	Flat-top guideway
Type II	Wrap around or clamp type guideway
Type III	Magnaplane guideway
Type IV	Invert "T" guideway
Type V	U-shaped or channel guideway

Three girder sections are chosen to design the five configurations. They are:

1. Box section - Type I, II, III, V
2. Double "T" section - Type I, II, III, V
3. Invert "T" section - Type IV

1.4 Maglev Guideway Structural Elements

The typical guideway structural system used in this study is shown schematically in Figure 1. It consists of the following structural elements:

1. Guideway girder
2. Column
3. Cantilever Arm
4. Spread footing

The guideway girder are designed to rest on two mechanically adjustable supports put on the top of the cantilever arms. The cantilever arms are small reinforced concrete beams cantilevered from the top of a free standing circular or rectangular column. The column is supported by a rectangular spread footing.

1.5 Computer Programs

The purpose of this study is to develop and write computer programs to do the Maglev guideway structural design. The procedure used in the design of each structural element will be computerized. A lot of unknowns such as the girder deflection, the girder span and the column height deserve special attention in the Maglev guideway design. They are treated as variables in computer programs. By using different values for the variables, the computer programs can help us design more efficient, economic and feasible guideway structures.

Two guideway programs are written. One uses the reinforced concrete girder and the other uses the prestressed concrete girder. It is the intent of the investigator to write user-friendly programs so that they can be performed without reading any input manual.

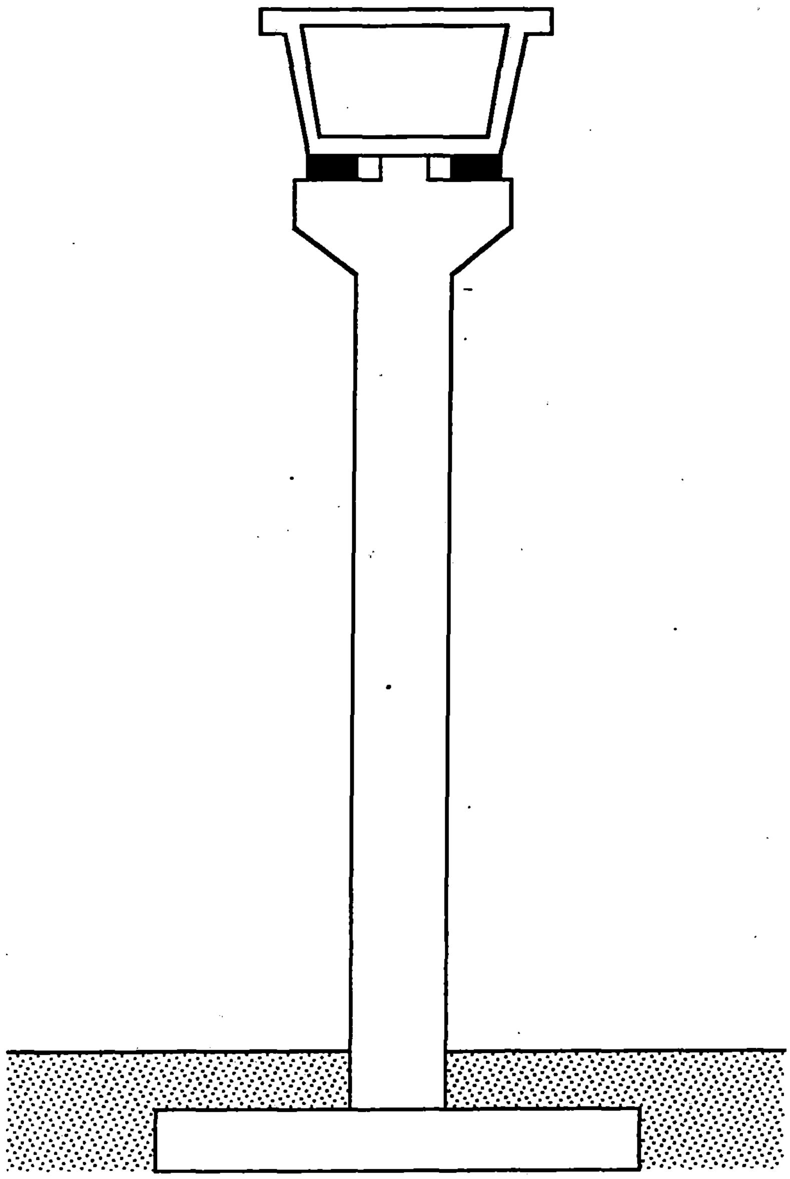


Figure 1 - Guideway Structural System

2. LOADS ON GUIDEWAY STRUCTURES

2.1 Dead Loads (D)

The dead load of a maglev structure consists of the weight of the entire structure, including the levitation strips, linear synchronous motor, power and control cables. In the analysis and design of the guideway girder, the dead is divided into:

(1) Girder dead load which depends on the section dimensions and span
(2) Additional (or superimposed) dead load which may include:

a. levitation strips and linear synchronous motors.
b. power and control cables
c. extra concrete placed on the girder
d. ice or snow

2.2 Live Loads (L)

The live load is due to the weight of maglev vehicles, which is estimated equal to 1,070 pounds per linear foot.

The live load should consist of the weight of one or more standard Maglev vehicles positioned to produce a maximum load effect in the structural element under consideration. However, since simply-supported girders are used in the guideway design and the magnets are evenly distributed at the bottom of the vehicles, it is assumed that a live load of 1,070 lb/ft is uniformly applied to the guideway girder.

2.3 Dynamic Effect of Vehicles (I)

It is well-known that a vehicle moving across a bridge at a normal rate of speed produces greater stresses than a vehicle that remains in a static position on the structure. This increment in stress can be called the dynamic effect or impact. AASHTO Section 3.8.2.1 uses the following equation to determine the "impact factor":

$$I = \frac{50}{125+L}$$

in which

I = impact factor (maximum 30 percent)
L = the length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

The stresses due to impact are calculated by multiplying the live load stress by the value of I. They are then added to the live load stresses to obtain the total stresses due to vehicle loads.

The above equation appears to be very conservative technically and structurally for the Maglev vehicles. Instead of touching and moving across the structure, they are levitated and travelling above the guideway. Also, due to the very small deflection requirement for the maglev guideway girders, maglev vehicles are moving across a much smoother girder than the highway bridge vehicles. In the present study, the impact factor is assumed to be 10 % of the live load (or 0.1 g).

Emergency impact factor is assumed to be 33% of the live load (or 0.33 g).

2.4 Longitudinal Forces (LF)

When the maglev vehicle brakes or accelerates longitudinal forces are transmitted from its wheels to the deck of the guideway girder. The magnitude of the longitudinal forces depends on the amount of acceleration or deceleration. the maximum longitudinal forces results from emergency braking of the vehicle. Due to the high velocity of maglev vehicles, the longitudinal force presented here is much more conservative than that presented in the Section 5.3.6 of ACI 358.1R-86. The longitudinal force is applied as follows:

Normal braking force =.	25 % of live load (0.25 g)
Emergency braking force =	100% of live load (1.0 g)
Normal acceleration force =	10% of live load (0.1 g)

2.5 Horizontal Vehicle Loads

The horizontal vehicle loads include the centrifugal force and the hunting force.

Centrifugal force (CF) - When a vehicle travels on a curvilinear path it produces a centrifugal force perpendicular to the tangent of the path. This force is given by

$$F = \frac{W}{g}\frac{v^2}{r}$$

where
W = weight of the vehicle,
g = acceleration of gravity (32.2 ft/s^2),
v = maximum velocity of the vehicle,
r = radius of the path of the vehicle.

In this study, the radius is assumed to be 1 mile.

Hunting force (HF) - The hunting force is a horizontal live load caused by the lateral interaction of the vehicle and the guideway. The hunting force is assumed to be 15% of the live load (0.15 g) as compared to the 8% used in the Section 5.3.5 of ACI 358.1R-86.

Since the hunting force is usually smaller than the centrifugal force, if centrifugal and hunting forces act simultaneously, only the larger force is considered (ACI 358.1R-86).

2.6 Wind Loads (WL)

The wind loads applied on the guideway superstructure and substructure are based on AASHTO Sections 3.15.1.1, 3.15.2.1.3, 3.15.2.2 and Section 5.5.1.3 of ACI 343R-88. They can be used for guideway girders not exceeding 200 ft. (60 m) in span. The loads specified here are based on a wind speed of 100 mph.

(a) For superstructure:

W_h (horizontal wind load on structure)
50 pounds per square foot but not less than 300 pounds per linear foot of the structure span length.

W_L (wind load on live load)
100 pounds per linear foot acting at 6 ft. above the deck

(b) For substructure:

Forces from superstructure

W_h 50 pounds per square foot, transverse;
12 pounds per square foot, longitudinal;
Both forces shall be applied simultaneously.

W_L 100 pounds per linear foot, transverse;
40 pounds per linear foot, longitudinal;
both forces shall be applied simultaneously.

Forces on substructure

W_h 40 pounds per square foot, transverse and longitudinal

2.7 Earthquake Forces (EQ)

A rigorous analysis of the effect of an earthquake on a structure is complex. It involves the application of structural dynamics. In addition, it is necessary to consider the relationship of the structure to the active faults and the seismic response of the soil under the substructure. The usual procedure is to simplify the problem by considering that the earthquake produces lateral forces acting in any direction at the center of gravity of the structure and having a magnitude equal to a percentage of the weight of the structure or any part of the structure under consideration. These lateral loads are then treated as static loads applied to the structures. The Equivalent Static Force method presented in AASHTO Section 3.21.1 is used to design the guideway structures to resist earthquake motions.

According to AASHTO Section 3.21.1, for structures with supporting members of approximately equal stiffness, an equivalent horizontal forces, EQ, may be applied to the structure. EQ is computed as

$$EQ = C\,F\,W$$

where

EQ = equivalent static horizontal force applied at the center of gravity of the structure;
F = framing factor;
F = 1.0 for structures where single columns or piers resist the horizontal forces;
F = 0.8 for structures where continuous frames resist horizontal forces along the frame;
W = total dead weight of the structure in pounds.
C = combined response coefficient determined from Figures 2 to 5.

The evaluation of C requires the computation of a natural period of vibration T of the structure and the determination of the maximum expected rock acceleration A. The period of vibration is computed by

$$T = 0.32\sqrt{\frac{W}{p}}$$

where p = total uniform force, pounds to cause a 1-inch maximum horizontal deflection of the whole structure. The rock acceleration A are determined as follows:

A = 0.09 g for zone I,
A = 0.22 g for zone II,
A = 0.50 g for zone III.

The zone classifications are obtained from Figures 6 to 8. Certain local jurisdictions have Zone IV high seismic risk requirements for analysis and design. For structures in this zone, response curves with A = 0.70 g in Figures 2 to 5 are used.

2.8 Loading Combinations

The guideway structure may be subjected to several loads simultaneously. The engineer has to decide what combination of loads and magnitudes will most likely be applied at one time. The AASHTO Section

3.22 has set up combinations of loads to be used in the serviceability design and strength design of highway bridge structures. The load combinations used in guideway structures also can be found in ACI 358.1R-86 (Analysis and Design of Reinforced Concrete Guideway Structures). The load combinations used in this study are based on ACI 358.1R-86. It is pointed here that the live load factor used in ACI 358.1R-86 is more reasonable than that in AASHTO since the live load is defined by the standard vehicle.

The strength design method is used in the Maglev guideway design and the following three strength-load combinations, U0, U1 and U2, are used:

U0 = 1.3 D + 1.7 (L + I + CF or HF)
U1 = 1.3 D + 1.4 (L + I + CF or HF) + 1.5 WL
U2 = 1.3 D + 1.4 (L + I + CF or HF) + 1.5 EQ + 1.4 LF (emergency braking)

The factored load to be used in the strength design is the maximum value calculated for the above combinations.

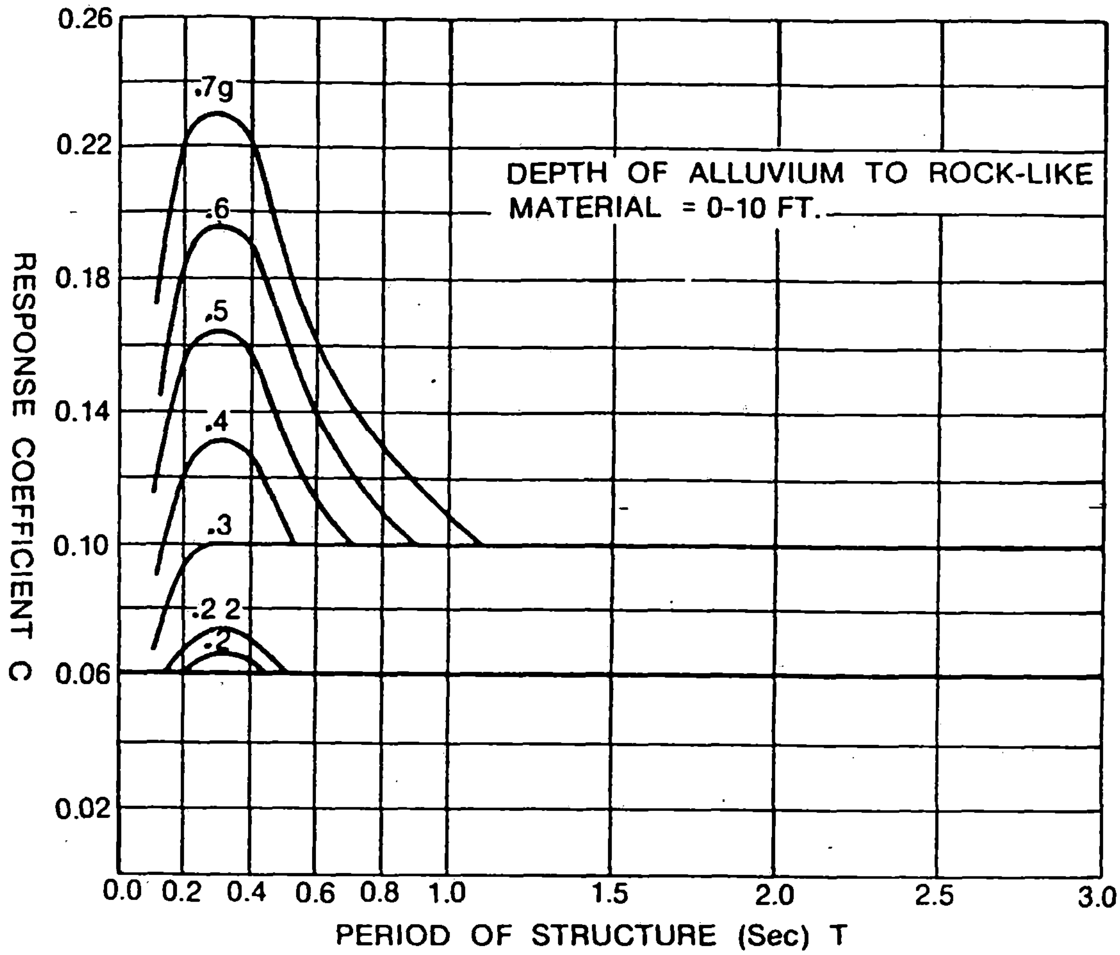


Figure 2 - Response Coefficient "C" for Various Values of Peak Rock Acceleration "A"

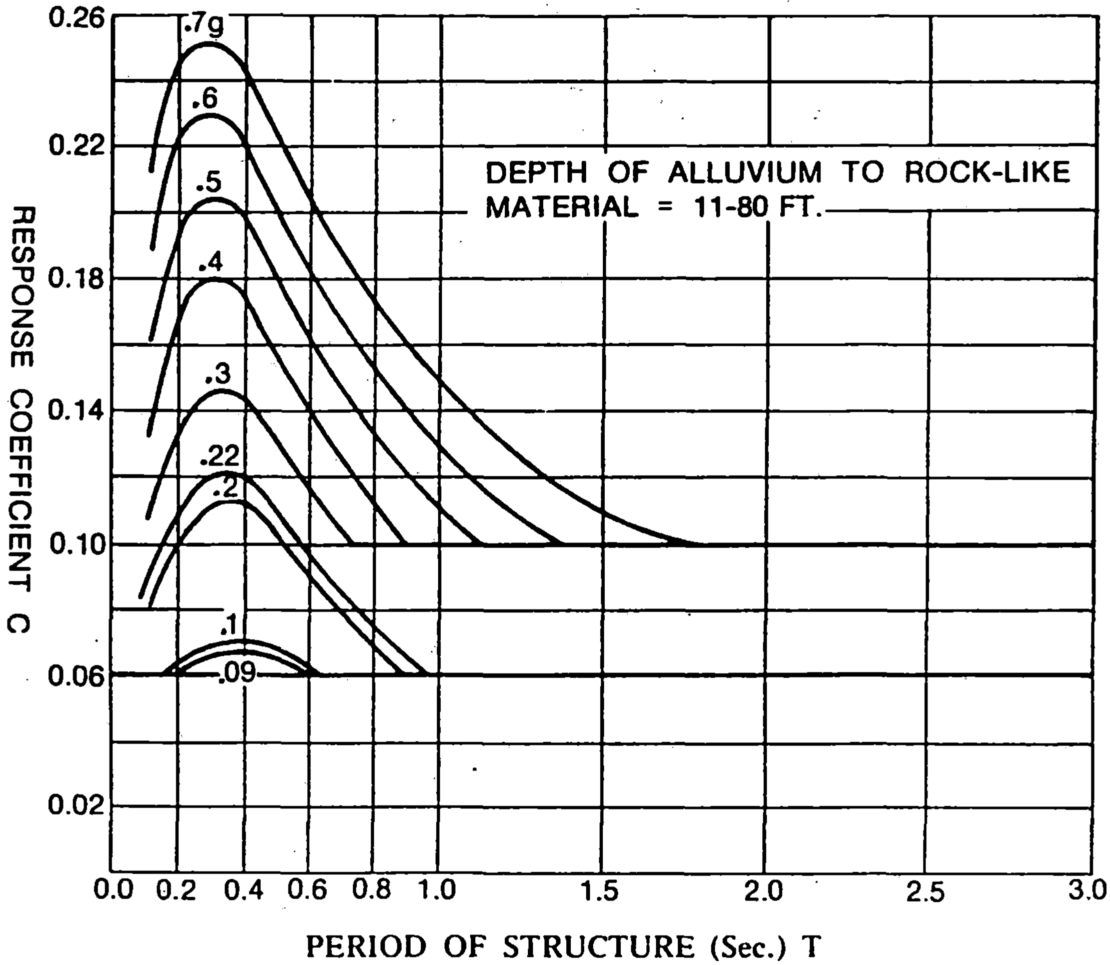


Figure 3- Response Coefficient "C" for Various Values of Peak Rock Acceleration "A"

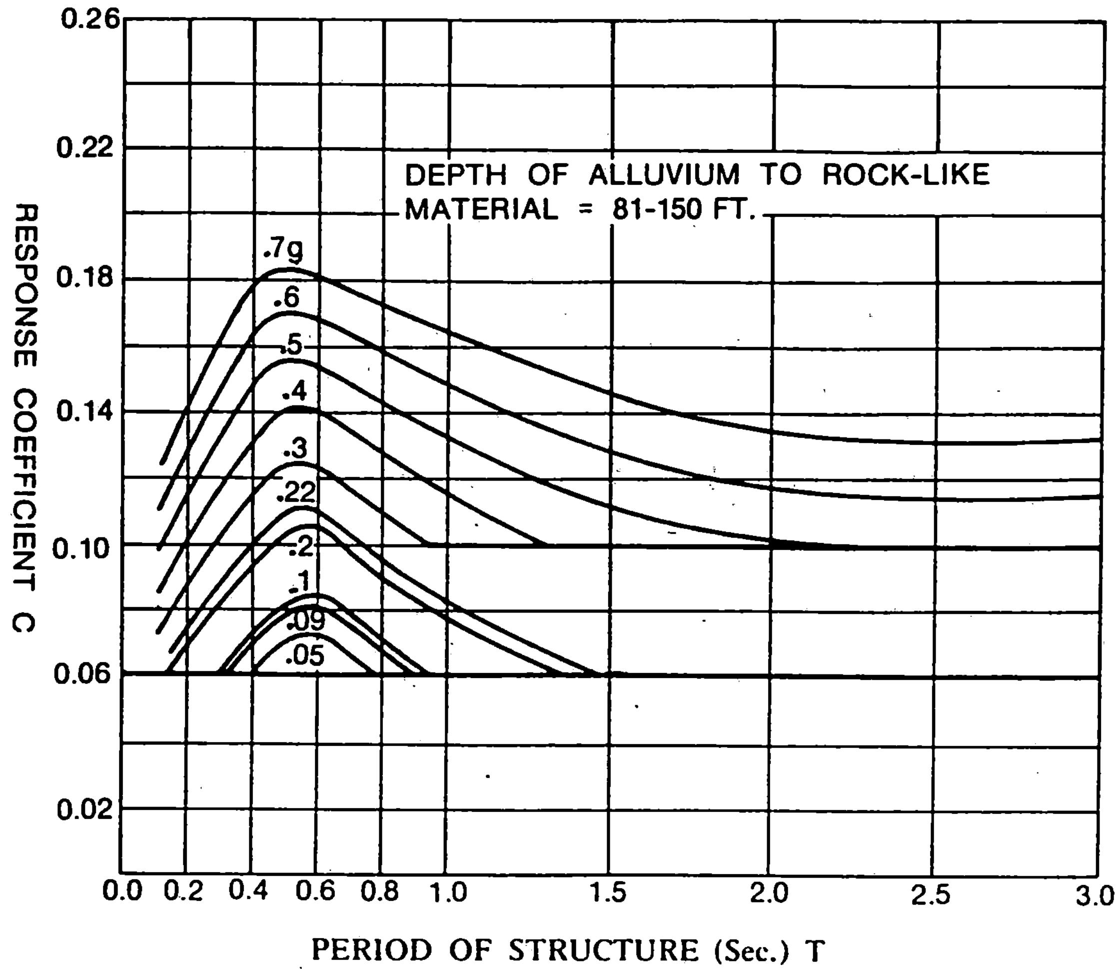


Figure 4 - Response Coefficient "C" for Various Values of Peak Rock Acceleration "A"

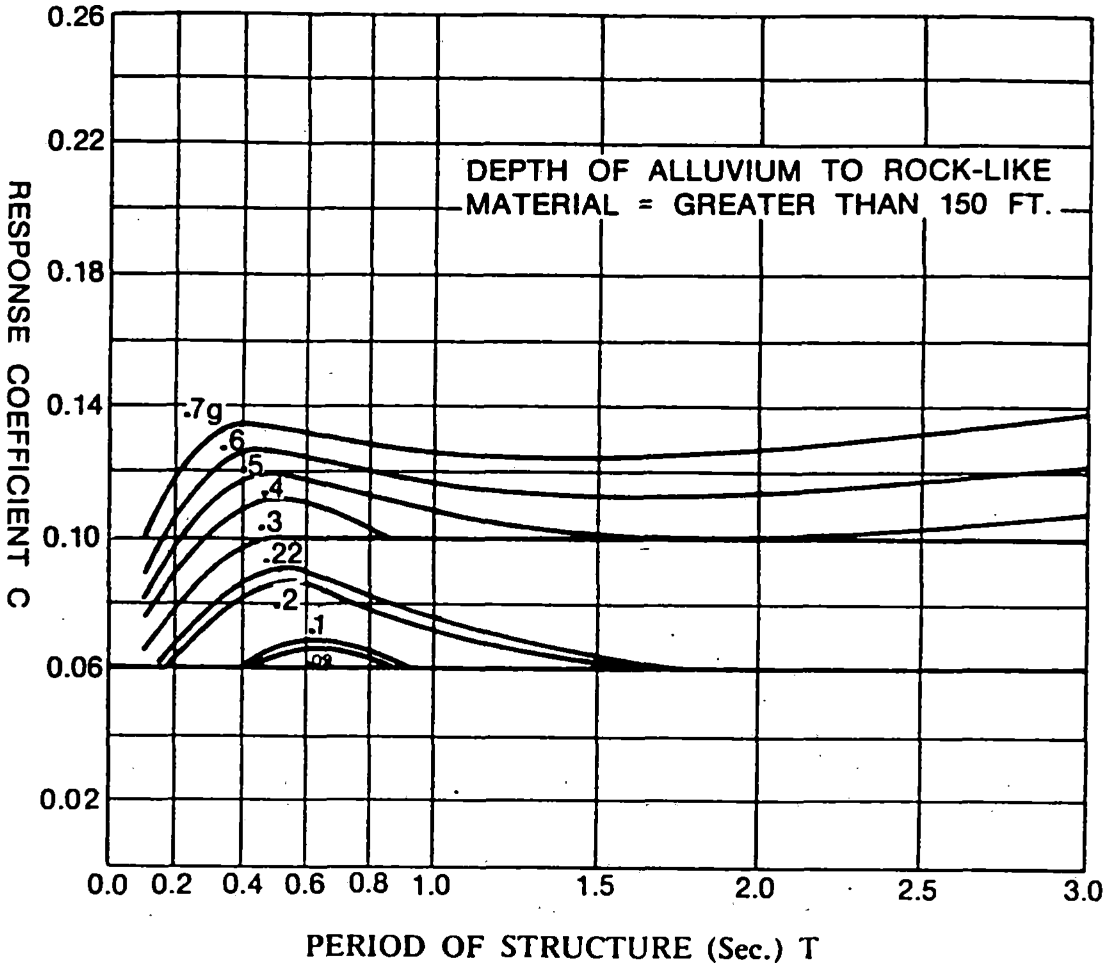


Figure 5 - Response Coefficient "C" for Various Values of Peak Rock Acceleration "A"

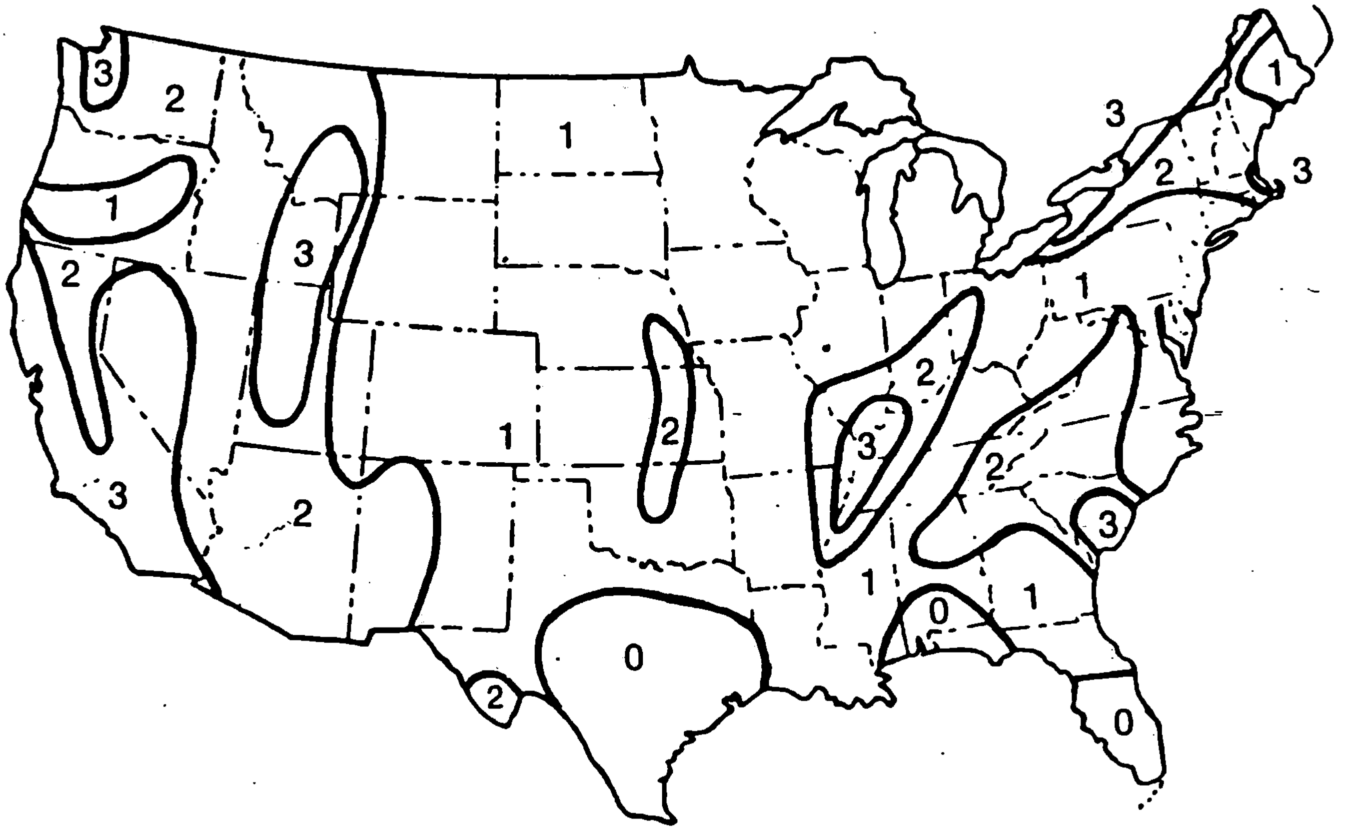


Figure 6 - Seismic Risk Map of the United States

3. DESIGN OF GUIDEWAY GIRDERS

Two types of concrete girders are used for the guideway. One is reinforced with the traditional steel rebars and the other is prestressed with seven-wire standard strands. Each type can have the following three different sections:

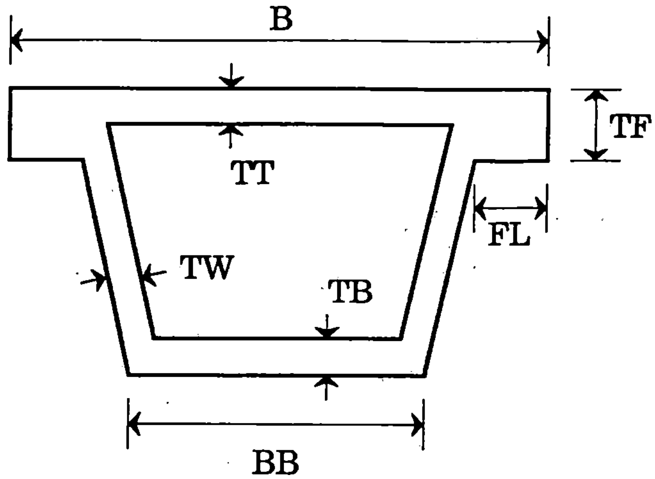
1. Box sections - Box girder construction contains a top slab that is usually wider than the box shape, which consists of vertical webs, and a bottom slab usually as broad as the out-to-out width of the girder web.
2. Double-T section - Double-T girder construction consists of two vertical rectangular stems with a wide top flange.
3. Invert-T section - Invert-T girder construction consists of a rectangular beam positioned in the middle of a box section with the top slab of the box section at about half the height of the beam.

The three typical sections used in the guideway girder design are shown in Figures 7 to 9. When they are applied to the five guideway configurations, Type I, II, III and V can use the box and double-T sections while Type IV can use the invert-T section.

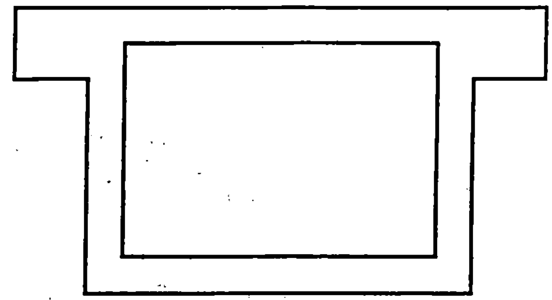
The dimensions of the girder sections are based on the structural considerations and the requirements of the levitation and propulsion systems. They are determined or assumed before starting the design. However, some of them, especially the proper depth of the girder, are modified during the design process. The flange, web or slab thickness used in the girder design for any type of sections must satisfy the requirements specified in Section 6.6 of ACI 343R-88 (Analysis and Design of Reinforced Concrete Bridge Structures). The default dimensions of the girders used in the computer programs are shown in Figure 10.

All of the guideway configurations are designed as simply supported beams. This means, as explained earlier, to have the metal reinforcing in the lower potion of the girder in order to reduce its interference of the magnetic field. Steel placed in the reinforced concrete girder satisfies such a requirement. However, for the prestressed concrete girder, this requires to design the girder with constant tendon eccentricity instead of variable tendon eccentricity. The design of guideway girders is much more complex than the traditional girders. One of the major reasons is that the girder deflection has to be considered in the design procedure. The guideway girder deflection is limited to about 1/2000 or 1/4000 of the girder span. The general design procedures for the reinforced concrete and prestressed concrete girders are described below.

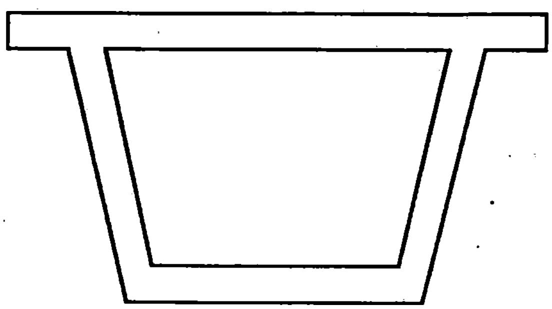
Figure 7 - BOX GIRDER SECTIONS



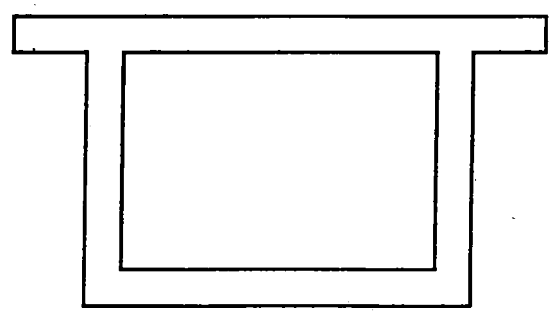
(A) Standard Section



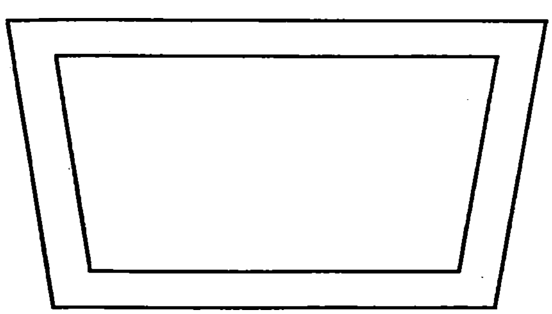
(D) If BB = B - 2xFL



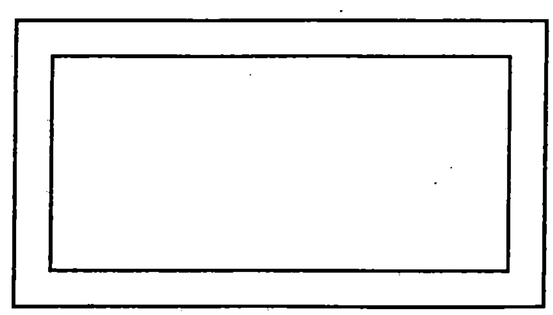
(B) If TF = TT



(E) If TF = TT and BB = B - 2xFL

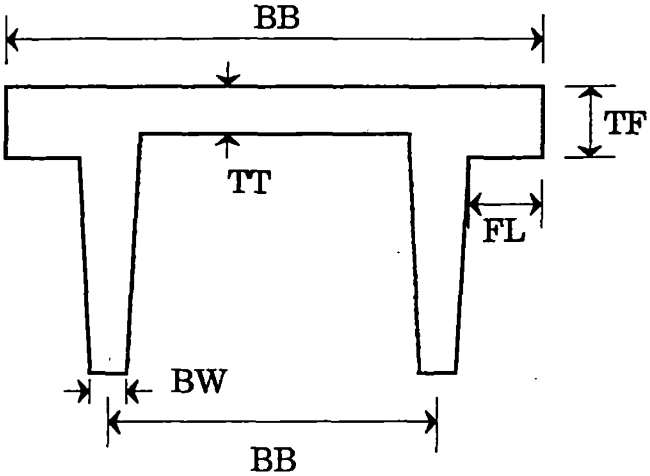


(C) If FL = 0

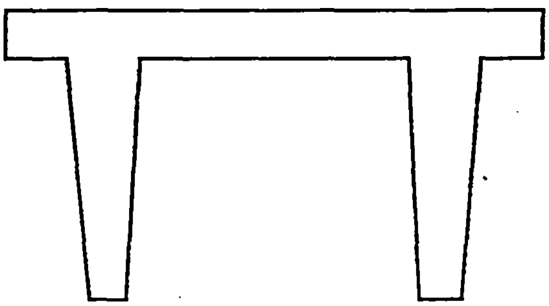


(F) If FL = 0 and B = BB

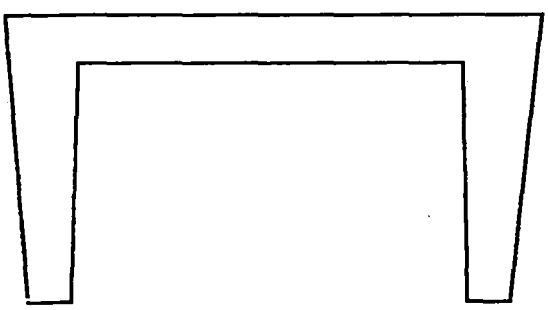
Figure 8 - DOUBLE-T GIRDER SECTIONS



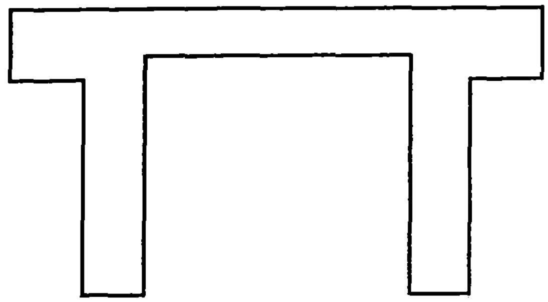
(A) Standard Section



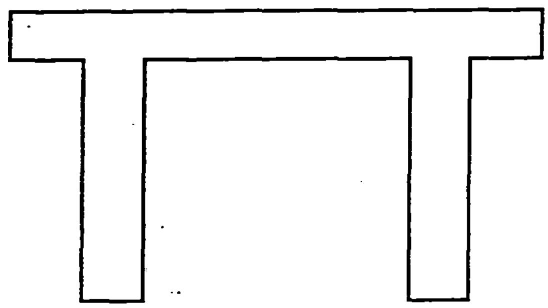
(B) If TF = TT



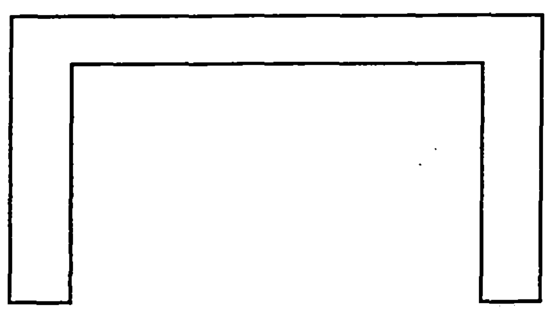
(C) If FL = 0



(D) If B = BB + BW + 2xFL

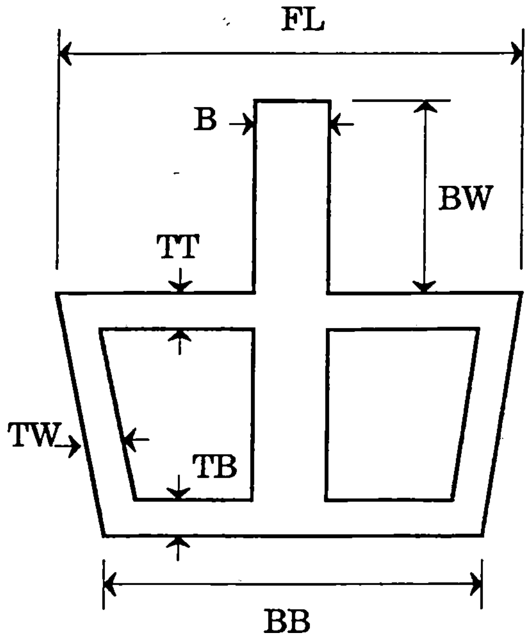


(E) If TF = TT and
B = BB + BW + 2xFL



(F) If FL = 0 and B = BB + BW

Figure 9 - INVERT-T SECTIONS



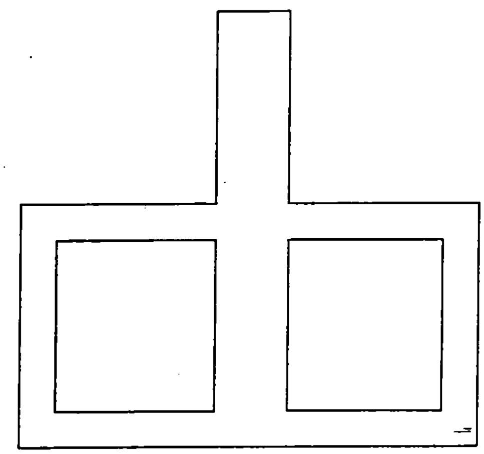
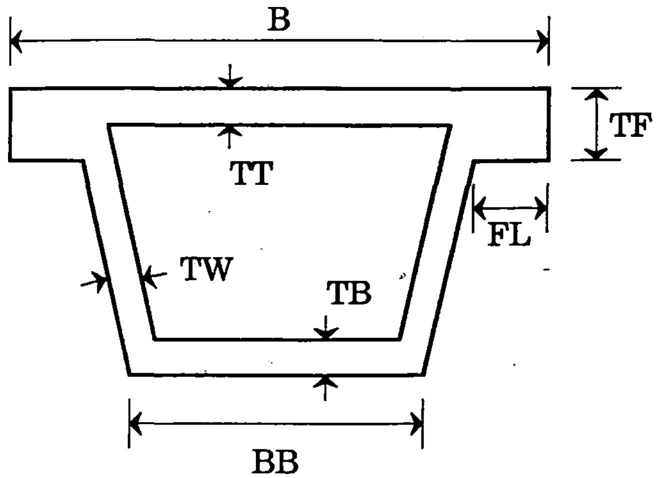
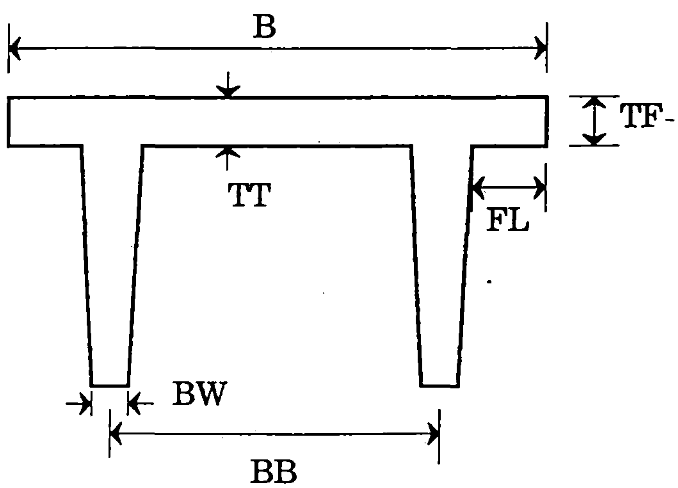


Figure 10 - DEFAULT GIRDER SECTION DIMENSIONS



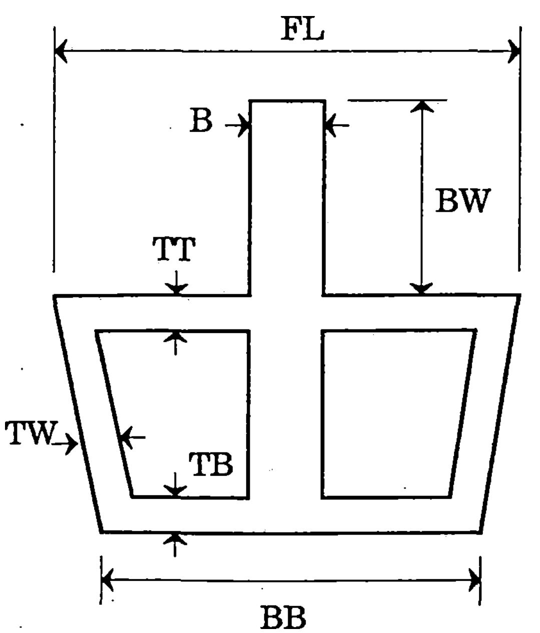
BOX GIRDER SECTION

B = 120 inches
BB = 84 inches
TT = 6 inches
TB = 6 inches
TW = 8 inches
FL = 12 inches
TF = 12 inches



DOUBLE-T GIRDER SECTION

B = 120 inches
BB = 84 inches
TT = 6 inches
BW = 10 inches
FL = 12 inches
TF = 6 inches



INVERT-T GIRDER SECTION

B = 12 inches
BB = 84 inches
TT = 6 inches
TB = 6 inches
TW = 6 inches
FL = 96 inches
BW = 42 inches

3.1 Reinforced Concrete Girder Design

DEFLECTION - In reinforced concrete beam design, deflection is inversely proportional to the girder's concrete modulus of elasticity, E_c and its effective moment of inertia, I_e. To achieve very small deflection, in addition to using high strength concrete, the effective moment of inertia has to be increased. I_e is expressed as (AASHTO Section 8.13.3 or ACI 318-89 Section 9.5.2.3)

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (1)$$

where
- M_{cr} = cracking moment = $f_r I_g / y_t$
- M_a = maximum moment in the member at the loading stage for which moment inertia is being calculated.
- f_r = modulus of rupture
- y_t = distance from centroid to extreme tension fiber
- I_g = gross moment of inertia
- I_{cr} = cracked moment of inertia

The above equation indicates that very small girder deflection can be obtained by increasing the gross moment of inertia, the cracked moment of inertia or both. The gross moment of inertia can be increased by having a larger girder section while the cracked moment of inertia by having more steel reinforcement in the section. When calculating the deflection, only the live load deflection is considered since the dead load deflection can be offset by cambering the girder during the construction. The live load deflection for the simply-supported girder, δ, is calculated using the following equation:

$$\delta = \frac{5}{384}\frac{w_L L^4}{E_c I_e} \qquad (2)$$

where w_L = live load per unit length and L = girder span

REINFORCEMENT - To prevent steel reinforcement from interfering the magnetic field, main steel reinforcement is placed at the bottom of the girder and no compressive steel is allowed at the top of the girder. This also leads to another assumption used in the design. That is, whether it is a box or a double-T girder, the compressive stress area lies entirely within the thickness of the top girder slab (the top slab minimum thickness stated in codes usually satisfies this requirement). If this is not the case, the thickness will be adjusted accordingly during the design. Thus, just like in a singly reinforced rectangular beam design, the following equation relating moment, beam geometry and steel ratio together is used in the design:

$$\frac{M_n}{bd^2} = \rho\, fy\left(1 - \frac{1}{2}\rho m\right) \tag{3}$$

Where
- b = girder width
- d = effective depth (distance from top of girder to steel reinforcement)
- F_y = steel yield strength
- m = $f_y / 0.85\, f_c'$
- ρ = steel ratio
- M_n = nominal moment capacity = $M_u/0.9$

M_n is determined from the factored loads.

DESIGN PROCEDURE - A procedure that can assure that the design solution satisfies the deflection and code requirement has been developed. To find the proper section depth and steel reinforcement for the girder, two extreme solutions, which are called the upper-bound and the lower-bound solutions, are determined first. The upper-bound solution finds the minimum girder depth if maximum allowable steel ratio is used. The lower-bound solution is obtained by keeping increasing the section depth until a maximum depth that satisfies the deflection requirement is found. After finding the two extreme solution, any girder design solution with a specified section depth between the upper and lower bound solutions will satisfies the deflection and code requirements. Based on the specified section depth, from Equation 3, the required steel ratio can be determined.

UPPER-BOUND SOLUTION - To increase I_{cr}, the girder section has to be designed with a high steel ratio. However, the maximum allowable steel ratio is 75% of the balanced steel ratio (ρ_b) The upper-bound define the minimum depth to satisfy the allowable deflection if 75% of the balanced steel ratio is used. This is an upper-bound solution.

The solution first uses Equation 3 to find d and then h (section depth, $h = d + d_c$, d_c is the distance from the reinforcement centroid to the bottom of the girder) by assuming $\rho = 0.75\ \rho_b$. The depth at this time generally does not satisfy the very small deflection requirement due to very low depth (low I_g). thus h has to be increased (ρ is not changed). However, due to high steel ratio, I_{cr} can increase faster if h is increased. Thus, without increasing h too much, I_{cr} contribute more to the increase of I_e. The secant method is used to find the required depth that can satisfy the allowable deflection (the relation between h and deflection is nonlinear).

LOWER-BOUND SOLUTION - To increase I_g, the girder section has to be designed with low steel ratio. Because, according Equation 3, the deeper the girder is, the less the steel ratio is required. Thus, if ρ for a section is high, d (and then h) will be low and deflection will exceed the allowable limit, and if ρ is low enough , h is high and deflection will be much less than the limit. The lower-bound solution based on Equation 3 (d and ρ are dependent), using the secant method (the relation between ρ and deflection is nonlinear), generates the optimum steel ratio (thus the optimum depth) that can satisfy the allowable deflection requirement. This is the lower-bound solution. Although a section that is deeper (I_g is greater) than the optimum depth can be used as the lower bound solution, however, due to its low steel ratio, it may not satisfy the fatigue requirement and the crack control criteria.

GIRDER DESIGN SOLUTION - The girder design solution is a solution between the upper-bound and lower-bound solutions. Thus, the girder design depth has to be between the minimum depth (upper-bound) and the maximum or optimum depth (lower-bound). After specifying a girder design depth, the secant method is used (the relation between ρ and deflection is nonlinear) to find the required steel ratio that can satisfy the allowable deflection. Before the design solution is generated, the selection of steel bars and bar size have to be performed to determine the distance from the centroid of the main reinforcement to the bottom of the girder. The selection procedure may be iterated until the solution converged. Also, the obtained steel ratio has to satisfy the following minimum steel ratio requirement (ACI 343R Section 7.3.4).

$$\rho_{min} = \left[10 + \frac{(I_g/y_t)}{bd^2}\right]\left[\frac{(I_g/y_t)}{bd^2}\right]\frac{\sqrt{f_c'}}{f_y} \qquad (4)$$

After the design solution is obtained, the girder is checked if it satisfy the fatigue requirement (AASHTO Section 8.16.8.3) and the crack control criteria (AASHTO Section 8.16.8.4). If not, the steel area is increased until they are satisfied. Next, the skin reinforcement (ACI 343R Section 8.4.3) is designed to prevent the side face crack for deep girders. Finally, the shear reinforcement is designed for the girder (AASHTO Section 8.16.6).

3.2 Prestressed Concrete Girder Design

DEFLECTION - Deflection calculation is much easier for prestressed concrete girders than for reinforced concrete girders. For reinforced concrete girders, cracks are caused by tensile and shear stresses. However, for prestressed concrete girders, due to the compressive steel strands, cracks are rarely present. The prestressed guideway girders will be designed in such a way that there are no cracks in them. Thus, when using Equation 2 to calculate live load deflection, the gross moment of inertia (I_g) is used instead of the effective moment of inertia (I_e).

$$\delta = \frac{5}{384}\frac{w_L L^4}{E_c I_g} \qquad (5)$$

REINFORCEMENT - To prevent steel strands from interfering the magnetic field, the prestressed guideway girder is design with constant tendon eccentricity. Girders with constant tendon eccentricity are girders with straight tendons located below its neural axis. no compressive steel is allowed at the top of the girder. Like in the reinforced concrete girder design, the prestressed concrete girder design also assumes that the compressive stress area lies entirely within the thickness of the top girder slab (the top slab minimum thickness stated in codes usually satisfies this requirement) whether it is a box or double-T section. If this is not the case, the thickness will be adjusted accordingly during the design.

MINIMUM SECTION DEPTH - Due to the interaction between the concrete girder and the prestressed and eccentric tendons, a minimum girder section depth must be provided to prevent any crack at top of a prestressed girder. For girders with constant tendon eccentricity, because the tendon has a large eccentricity at support, creating large tensile stresses at the top fiber without any reduction due to superimposed M_D + M_{SD} + M_L. In other words, the support section is the control section in the girder design. It can be shown that the minimum section depth is the depth that can provide the following section moduli values:

$$S_t = \frac{M_D + M_{SD} + M_L}{\gamma f_{ti} - f_c} \tag{6}$$

$$S_b = \frac{M_D + M_{SD} + M_L}{f_t - \gamma f_{ci}} \tag{7}$$

where S_t = section modulus for the top fibers
S_b = section modulus of the bottom fiber
M_D = moment due to girder self-weight
M_{SD} = moment due to superimposed dead load
M_L = moment due to live load, including impact
γ = residual prestress ratio

DESIGN PROCEDURE - A procedure presented by Nawy (Prestressed Concrete - A fundamental Approach, 1988) was adopted in this study. However, it is modified to include the deflection requirement. The modified procedure can assure that the design solution satisfies the deflection and code requirement .

The procedure first performs the service-load design of the prestressed girder and then check if the girder satisfies the ultimate-load design. The general procedure is shown below:

1. Determine the minimum section depth

   The minimum girder section depth is the least of (a) the minimum section depth that provides the minimum section modulus for the girder based on superimposed M_D + M_{SD} +M_L, or (b) the minimum section depth that satisfies the deflection requirement.

2. Choose a section with its depth greater than the minimum section depth

3. Based on the chosen section, compute the prestressed force, eccentricity, and required prestressed area.

$$P_i = A_c f_{ci} \tag{8}$$

$$f_{cic} = f_{ti} - \frac{c_t}{h}(f_{ti} - f_{ci}) \tag{9}$$

$$e_e = (f_{ti} - f_{cic})\frac{S_t}{P_i} \tag{10}$$

$$A_p = \frac{P_i}{f_{pi}} \tag{11}$$

where
P_i = initial prestressing force
A_c = girder section area
f_{ci} = maximum allowable compressive stress in concrete immediately after transfer and prior to losses
f_{cic} = concrete stress at transfer at the level of the centroid of the concrete section
f_{ti} = maximum allowable tensile stress in concrete immediately after transfer and prior to losses
c_t = the distance from the center of gravity of the section to the extreme top fiber
h = girder section height
e_e = the distance from the centroid of the girder section to the centroid of steel strands
S_t = the section modulus for the top fiber
A_p = prestressed steel area
f_{pi} = the initial prestress

4. Check for girder section fiber stress requirements at critical sections.

   If any of the stress requirements is not satisfied, a greater section depth has to be chosen. Thus, go to step 2.

5. Perform ultimate-strength design of the prestressed girder.

   Check if the provided nominal moment capacity (M_n) is greater than the required nominal moment capacity ($M_u/0.9$).

4. REINFORCED CONCRETE SUBSTRUCTURE DESIGN

Substructure design is significant because of the many possible load variations that can exercise control. Ordinarily most loading combinations are important only in the design of the substructure. The substructure must be designed to accommodate adequate resistance to vertical and lateral loads and be designed to prevent any serious settlement that might cause distress to any of the portions of the superstructure. In the design of the reinforced concrete substructure units the details of three basic components are presented: (1) columns, (2) cantilevered arms, and (3) spread footings.

4.1 Columns

The column configuration considered in this study is either circular or rectangular. Reinforced concrete rectangular columns are more efficient than circular column when designed to carry moment. Circular columns have some advantages, such as ease in forming and the added confining strength that results from spirals incorporated in the section. The close spacing of spiral reinforcing also provides excellent buckling strength characteristics to the main reinforcing, a definite advantage under heavy earthquake loading.

COLUMN MOMENTS AND AXIAL LOADS - The bending moments (M_{ux}, M_{uy}) and axial load (P_u) are calculated based on the factored loads applied to the column. However, due to that the guideway girders supported by the columns have very high ground clearance, the moments acting on the columns must be magnified to consider the column slenderness effect. The moment magnifier method for designing slender columns presented in ACI Sec. 10.11 is used.

According to ACI Section 10.11.4, for unbraced columns, slenderness effects must be included in column analysis or design if

$$\frac{KL_u}{r} > 22 \qquad (12)$$

where K = column effective length factor
L_u = unsupported length of column
r = radius of gyration

for a rectangular cross section, r = 0.3 h, and for a circular section, r= 0.25h. Due to that the guideway columns designed in this study are cantilevered and unbraced, K = 2 is used.

ACI Section 10.11.5.1 states that columns shall be designed for the factored axial load, P_u, and a magnified factored moment, M_c. The M_c for the columns in this study cab be defined by

$$M_c = \delta_s M_{2s} \tag{13}$$

where M_{2s} is the larger factored end moment due to load which result in appreciable sidesway and δ_s is the moment magnification factor for unbraced columns. It is calculated from the following equation:

$$\delta_s = \frac{1}{\left(1 - \frac{P_u}{\phi P_{cr}}\right)} \tag{14}$$

where

$$P_{cr} = \frac{\pi^2 EI}{(Kl_u)^2} \tag{15}$$

and

$$EI = \frac{E_c I_g/5 + E_s I_{se}}{1 + \beta_d} \tag{16}$$

or

$$EI = \frac{E_c I_g/2.5}{1 + \beta_d} \tag{17}$$

$$\beta_d = \frac{|\,M_2 \text{ due to factored dead loads}\,|}{|\,\text{total factored } M_2\,|} \tag{18}$$

In the above equations,

ϕ = strength reduction factor
P_{cr} = the critical load
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of steel
I_g = gross moment of inertia of concrete section about its centroidal axis ignoring the reinforcement
I_{se} = moment of inertia of the reinforcement about the centroidal axis of the concrete section.

For biaxially loaded columns, M_c for each bending axis is calculated.

RECTANGULAR COLUMN SECTION ANALYSIS - Due to that not only the longitudinal forces but also transverse forces act on the guideway structure, the guideway column is subjected to axial loads accompanied by bending about two perpendicular axes. For the analysis of biaxially loaded rectangular columns, ACI recommended two methods, the Reciprocal Load method and the Load Contour method. The column analysis procedure used in this study follows AASHTO Section 8.16.4.3:

In lieu of a general section analysis based on stress and strain compatibility, the design strength of noncircular members subjected to biaxial bending may be computed by the following approximate expressions:

$$\frac{1}{P_{nxy}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_o} \qquad (19)$$

when the factored axial load,

$$P_u \geq 0.1 f_c' A_g \qquad (20)$$

or

$$\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \leq 1 \qquad (21)$$

when the factored axial load,

$$P_u < 0.1\, f_c' A_g \qquad (22)$$

Equation 19 represents the Reciprocal Load method while Equation 21 is similar to the equation used in the Load Contour method.

RECTANGULAR COLUMN SECTION DESIGN - Basically the rectangular biaxially loaded column design is a trial and error procedure. A trial solution has to satisfy Equation 19 or Equation 21. If not, another trial solution is checked until a proper solution is found. Equations 19 and 21 require the use of the rectangular column interaction diagrams.

CIRCULAR COLUMN SECTION DESIGN - For round columns, biaxial bending does not require special design methods because resistance about any axis is the same. One simply designs for $\sqrt{M_x^2 + M_y^2}$, where M_x and M_y are moments about the x and y axes. The required steel reinforcement can be found from the proper round column interaction diagram.

One of the major tasks of this study is to computerize all the column interaction diagrams so that they can be used in the biaxially loaded column design.

4.2 Cantilever Arms

The strength design procedure for a singly reinforced concrete beam is used to design the cantilever arms. The moments acting on the arms are calculated by assuming that loads transmitted from the guideway girders are uniformly distributed over the contact area at the top of the substructure where the girders are supported.

4.3 Spread Footings

Spread footings are square or rectangular pads which spread the axial load and bending moments from the column over an area of soil that is large enough to support them. The soil pressure causes the footing to deflect upward, causing tension in the longitudinal and transverse directions at the bottom of the footing. As a result, reinforcement is placed in the two directions at the bottom.

DISTRIBUTION OF SOIL PRESSURE UNDER A FOOTING - The soil pressure under a footing is calculated assuming linearly elastic action in compression but no tensile strength across the contact between the footing and soil. For columns with biaxial bending, if the column load is applied at, or near, the middle of the footing, the pressure, q, under the footing is

$$q = \frac{P}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \qquad (23)$$

where P = vertical axial load, positive in compression
A = area of the footing
I_x = moment of inertia of this area about the centroidal x axis
I_y = moment of inertia of this area about the centroidal y axis
M_x = moment about the centroidal x axis of the area
M_y = moment about the centroidal y axis of the area
x = distance from the centroidal y axis to point where the pressures are being calculated
y = distance from the centroidal x axis to point where the pressures are being calculated

The moment, M_x, can be expressed as Pe_y, where e_y is the eccentricity of the load relative to the centroidal x axis of the area A. The moment, M_y, can be expressed as Pe_x, where e_x is the eccentricity of the load relative to the centroidal y axis of the area A. If M_x is equal to zero, one can find the maximum eccentricity, ey, is equal to L/6, and if M_y is equal to zero, one can find the maximum eccentricity, e_x, is equal to B/6. The maximum eccentricities, e_x and e_y, are referred to as the kern distance. Loads applied within the kern, the shaded area in Figure 11, will cause compression over the entire area of the footing and the above equation can be used to compute the footing pressure, q.

If the load falls outside the kern, eccentricities will cause a portion of footing to lift off the soil, since the soil-footing interface can not resist tension. The footing pressure distribution under such a loading condition would be acceptable only if the maximum pressure under the footing is less than the allowable soil pressure and if the footing does not tilt excessively. Generally, however, it would not be acceptable, since it makes inefficient use of the footing concrete and tends to overload the soil. AASHTO Section 4.4.7.1.1.1 (Interim 1991) requires that footings on soil shall be designed so that the eccentricity of loading is less than 1/6 of the footing dimension in any direction. In the guideway footing design, if the above equation gives the minimum pressure under the footing a negative value, the size of the footing will be increased. However, if the increased size of the footing is too large and is not economically feasible, it is suggested that the designer use piles under the footing or extend the column deep into the soil.

SPREAD FOOTING DESIGN - It is generally the case that the allowable footing pressures are given at a service load level. Therefore the footing is proportioned for a size on service load. At the beginning of the footing design, first the service level loads and moments are calculated for the footing and then the actual pressures are computed and compare with the allowable footing pressure based on the assumed footing size. If the maximum pressure under the footing is great than the allowable footing pressure, the footing size has to be increased.

In the design of footing reinforcement, however, the factored load and moment are calculated and then used to compute the pressures under the footing. The pressures are then used for the footing reinforcement design.If the minimum footing pressure is negative (in tension), the footing size has to be increased.

After determining the footing pressures, the proceudre presented by Heins and Lawrie (Design of Modern Concrete Highway Bridges, 1984) is used to design the reinforcement in the footing. Based on the calculated footing pressures, the moments per foot are computed by cantilever action at the corner of the column shaft in the transverse as well as longitudinal directions. The strength design procedure for a singly reinforced concrete beam is then applied to the two directions to find the reinforcement for the footing.

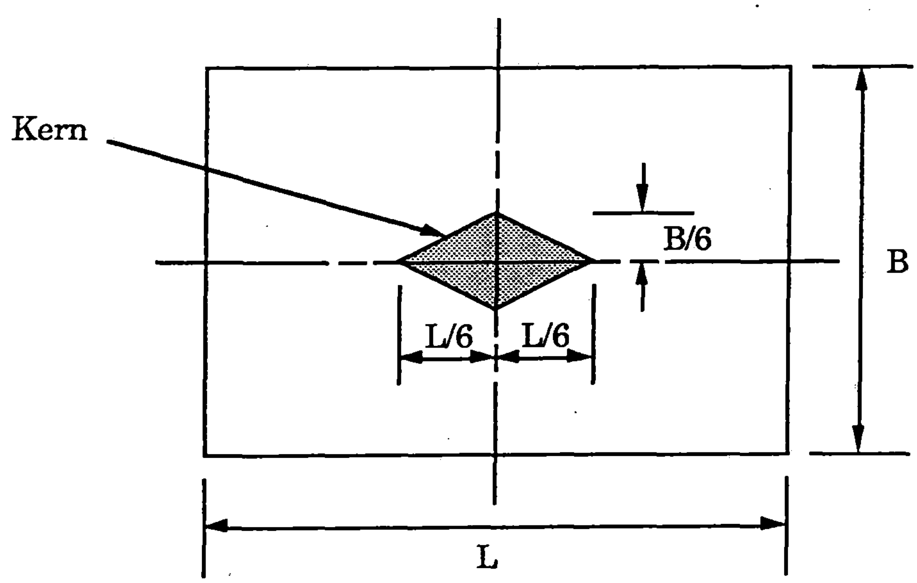


Figure 11 - Kern Dimensions

5. COMPUTER PROGRAMS

Two computer design programs, GWAY0 and GWAY1 have been written to perform the guideway structural design. GWAY0 uses the reinforced concrete girder for the guideway while GWAY1 uses the prestressed concrete girder. Each program has four design steps and performs the guideway structural design in the following sequence:

1. guideway girder design
2. column design
3. cantilever arm design
4. spread footing design

GWAY0 and GWAY1 are user-freindly computer programs. They are based on the text-user interface. Users only have to follow the text messages shown on the computer screen to run the programs. These messages usually instruct the user to input the design data or select a design option. The design solution will shown on the screen and save simultaneously in a file called OUTPUT.DAT. Four key design algorithm used in the two programs are described below.

5.1 Reinforced Concrete Girder Design Algorithm

1. Input design data:

   f_c' = specified compressive strength of concrete, in psi
   f_y = specified tensile strength of steel, in psi
   W_D = girder dead load per ft
   W_{SD} = superimposed dead load per ft
   W_L = vehicle weight per ft
   L = girder span, in ft
   δ_a = allowable girder deflection, in inches

2. Input guideway section dimensions (Assume initial girder depth is 60 inches).

3. calculate the balanced steel ratio, ρ_b

4. find lower-bound solution - use the secant method to determine the lowest steel ratio that can provide a deeper girder depth.

5 find upper-bound solution - use the secant method to determine the minimum girder depth using the maximum allowable steel ratio, $\rho = 0.75\ \rho_b$.

6 Enter the girder design depth. The chosen girder design depth should be greater than the minimum section depth so that the girder will have a live load plus impact deflection less than the allowable deflection.

7. find the steel ratio for the girder with the specified design depth

8. Select the steel bar size and arrangement to be used for the girder

9. Check if the reinforcement satisfies the minimum steel ratio requirement.

10. check fatigue requirement

11. check crack control

12. design solution output

13. compute skin reinforcement requirement

14. compute shear reinforcement

15. compute slab reinforcement

5.1.1 Discussion on reinforced concrete girder design

1. The Invert-T section girder design is essentially a rectangular beam design. Due to that the section is so deep, the depth difference between the lower bound and upper bound solution is very small (about 10 inches).

2. The strength design method is used in the girder main reinforcement design because the ductile steel is used. However, since the FRP rebar is not a ductile material, the working stress method is used.

3. Due to that the behavior between the FRP rebar and the concrete is still not completely understood, in the FRP reinforcement design, the FRP rebar yield stress is conservatively set at 100,000 psi. The allowable rebar stress is 30% of the yield stress (30,000 psi) and the allowable concrete stress is 30% of concrete strength (1,800 psi). In AASHTO, the allowable steel rebar stress is 40% of the yield stress and the allowable concrete stress is 40% of the concrete strength (AASHTO Section 8.15.2).

4. For the same reason stated above, the concrete cover in the slab deck design is conservatively set at 2 inches. In AASHTO, the cover for the top reinforcement is 2 inches and for the bottom reinforcement 1 inches (AASHTO Section 8.22.1)

5. If the design height is greater than the height in the suggested lower bound solution, the design solution is going to be very conservative. If this is the case, the program computes the required steel ratio and then check the minimum steel ratio , the fatigue and the crack control requirements. The girder deflection for this solution is usually much less than the allowable deflection.

5.1.2 Output example #1

```
#####################
#  PROGRAM GWAY-91  #
#####################


==============================================================================
INPUT GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = BOX GIRDER
CONCRETE STRENGTH =      6000.0 PSI
STEEL STRENGTH =     60000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> BOX GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
WEB THICKNESS (TW) =   8.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
FLANGE LENGTH (FL) =  12.0 INCHES
FLANGE THICKNESS (TF) =  12.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> SUGGESTED LOWER BOUND SOLUTION <<<
OPTIMUM STEEL RATIO =   .0020505
OPTIMUM STEEL AREA =     19.916 IN**2
OPTIMUM(MAXIMUM) GIRDER DEPTH =     83.937 INCHES
GIRDER TOP SLAB THICKNESS =       6.000 INCHES
GIRDER L+I DEFLECTION =   .6002750 INCHES

>>> UPPER BOUND SOLUTION <<<
MAXIMUM STEEL RATIO =   .0282972
MAXIMUM STEEL AREA =   151.393 IN**2
MINIMUM GIRDER DEPTH =     47.584 INCHES
GIRDER TOP SLAB THICKNESS =     14.842 INCHES
GIRDER L+I DEFLECTION =   .6000000 INCHES
YC =  17.69489554 INCHES
CC =  20.12994126 INCHES

>>> CHECK FATIQUE REQUIREMENT <<<
LOGITUDINAL STEEL STRESS RANGE IS   6632.778 PSI.
ALLOWABLE FATIGUE STRESS RANGE IS  19515.484 PSI.
GIRDER FATIGUE REQIREMENT OK.

>>> CHECK CRACK CONTROL CRITERIA <<<
LOGITUDINAL STEEL MAXIMUM STRESS IS  18404.038 PSI.
```

```
ALLOW. MAX. STRESS FOR CRACK CONTROL IS  37463.806 PSI.
GIRDER CRACK CONTROL OK.

>>> FINAL BOX GIRDER DESIGN SOLUTION <<<
SPECIFIED GIRDER DEPTH =      60.000 INCHES
GIRDER TOP SLAB THICKNESS =       6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =      7.790 INCHES
GIRDER SECTION AREA =   2266.559 IN**2
GIRDER TOTAL VOLUME =   1574.000 FT**3
GIRDER DEAD LOAD DEFLECTION =  1.3011383 INCHES
GIRDER L+I DEFLECTION =   .5998544 INCHES
REQUIRED STEEL RATIO =   .0115023
REQUIRED STEEL AREA =     77.440 IN**2
PROVIDED STEEL RATIO =   .0115067
PROVIDED STEEL AREA =     77.470 IN**2
PROVIDE  61-# 10 IN  2 LAYERS

>>> COMPUTE SKIN REINFORCEMENT REQUIREMENT <<<
TOTAL SKIN REINFORCEMENT AREA =      1.582 IN**2
PROVIDE  4-#4 FRP BARS OVER EACH BOX WEB
AT A DISTANCE (28.053 INCHES) ADJACENT TO THE MAIN REINFORCEMENT.

>>> DESIGN GIRDER SHEAR REINFORCEMENT <<<
SHEAR REINFORCEMENT DESIGN SOLUTION
-----------------------------------------------------------------------------
#5 STIRRUP,   SPACING =   8.66INCHES,  FROM    .00 FT.   TO  10.00 FT.  4 LEGS
#5 STIRRUP,   SPACING =  12.27INCHES,  FROM  10.00 FT.   TO  20.00 FT.  4 LEGS
#5 STIRRUP,   SPACING =  19.57INCHES,  FROM  20.00 FT.   TO  30.00 FT.  4 LEGS
#5 STIRRUP,   SPACING =  24.00INCHES,  FROM  30.00 FT.   TO  40.00 FT.  4 LEGS
#5 STIRRUP,   SPACING =  24.00INCHES,  FROM  40.00 FT.   TO  50.00 FT.  4 LEGS
-----------------------------------------------------------------------------

>>> GUIDEWAY SLAB REINFORCEMENT DESIGN <<<
USE #4 FRP REBAR AT  9.00 INCHES FOR POSITIVE AND NEGATIVE MOMENTS
```

5.1.3 Output example #2

```
#####################
#  PROGRAM GWAY-91  #
#####################



==============================================================================
INPUT GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = DOUBLE-T GIRDER
CONCRETE STRENGTH =     6000.0 PSI
STEEL STRENGTH =    60000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN =  80.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> DOUBLE-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER BOTTOM THICKNESS (BW) =  10.0 INCHES
WEB SPACING (BB) =  84.0 INCHES
TOP SLAB THICKNESS (TT) =   8.0 INCHES
FLANGE LENGTH (FL) =  12.0 INCHES
FLANGE THICKNESS (TF) =   8.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> SUGGESTED LOWER BOUND SOLUTION <<<
OPTIMUM STEEL RATIO =   .0020153
OPTIMUM STEEL AREA =     15.478 IN**2
OPTIMUM(MAXIMUM) GIRDER DEPTH =      69.999 INCHES
GIRDER TOP SLAB THICKNESS =      8.000 INCHES
GIRDER L+I DEFLECTION =   .5999813 INCHES

>>> UPPER BOUND SOLUTION <<<
MAXIMUM STEEL RATIO =   .0282972
MAXIMUM STEEL AREA =    112.376 IN**2
MINIMUM GIRDER DEPTH =     39.094 INCHES
GIRDER TOP SLAB THICKNESS =     11.017 INCHES
GIRDER L+I DEFLECTION =   .6000000 INCHES
YC =  11.59762303 INCHES
CC =  14.91453475 INCHES

>>> CHECK FATIQUE REQUIREMENT <<<
LOGITUDINAL STEEL STRESS RANGE IS  10382.758 PSI.
ALLOWABLE FATIGUE STRESS RANGE IS  17849.296 PSI.
GIRDER FATIGUE REQUIREMENT OK.

>>> CHECK CRACKCONTROL CRITERIA <<<
LOGITUDINAL STEEL MAXIMUM STRESS IS  27203.072 PSI.
ALLOW. MAX. STRESS FOR CRACK CONTROL IS  32096.163 PSI.
```

```
GIRDER CRACK CONTROL OK.

>>> FINAL DOUBLE-T GIRDER DESIGN SOLUTION <<<
SPECIFIED GIRDER DEPTH =       60.000 INCHES
GIRDER TOP SLAB THICKNESS =        8.000 INCHES
GIRDER SECTION AREA =    2104.000 IN**2
GIRDER TOTAL VOLUME =    1168.889 FT**3
GIRDER DEAD LOAD DEFLECTION =  1.1788827 INCHES
GIRDER L+I DEFLECTION =    .5850095 INCHES
REQUIRED STEEL RATIO =    .0042507
REQUIRED STEEL AREA =      27.126 IN**2
PROVIDED STEEL RATIO =    .0043876
PROVIDED STEEL AREA =      28.000 IN**2
PROVIDE  14-#  9 IN  5 LAYERS IN ONE WEB

>>> COMPUTE SKIN REINFORCEMENT REQUIREMENT <<<
TOTAL SKIN REINFORCEMENT AREA =          1.331 IN**2
PROVIDE  4-#4 FRP BARS OVER EACH T-BEAM WEB FACES
AT A DISTANCE (26.590 INCHES) ADJACENT TO THE MAIN REINFORCEMENT.

>>> DESIGN GIRDER SHEAR REINFORCEMENT <<<
SHEAR REINFORCEMENT DESIGN SOLUTION
------------------------------------------------------------------------------
#4 STIRRUP,  SPACING =   9.80INCHES,  FROM    .00 FT.   TO   8.00 FT.  4 LEGS
#4 STIRRUP,  SPACING =  16.84INCHES,  FROM   8.00 FT.   TO  16.00 FT.  4 LEGS
#4 STIRRUP,  SPACING =  24.00INCHES,  FROM  16.00 FT.   TO  24.00 FT.  4 LEGS
#4 STIRRUP,  SPACING =  24.00INCHES,  FROM  24.00 FT.   TO  32.00 FT.  4 LEGS
#4 STIRRUP,  SPACING =  24.00INCHES,  FROM  32.00 FT.   TO  40.00 FT.  4 LEGS
------------------------------------------------------------------------------

>>> GUIDEWAY SLAB REINFORCEMENT DESIGN <<<
USE #4 FRP REBAR AT 12.00 INCHES FOR POSITIVE AND NEGATIVE MOMENTS
```

5.1.4 Output example #3

```
#####################
#  PROGRAM GWAY-91  #
#####################


==============================================================================
INPUT GIRDER DESIGN DATA
==============================================================================


CONCRETE STRENGTH =      6000.0 PSI
STEEL STRENGTH =    60000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> INVERT-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) =  12.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
SIDE WEB THICKNESS (TW) =   6.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
TOP SLAB WIDTH (FL) =  96.0 INCHES
HEIGHT BETWEEN TOP SLAB AND GIRDER TOP (BW) =  42.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> SUGGESTED LOWER BOUND SOLUTION <<<
OPTIMUM STEEL RATIO =   .0150319
OPTIMUM STEEL AREA =     18.328 IN**2
OPTIMUM(MAXIMUM) GIRDER DEPTH =   104.609 INCHES
GIRDER TOP SLAB THICKNESS =      6.000 INCHES
GIRDER L+I DEFLECTION =   .6000054 INCHES

>>> UPPER BOUND SOLUTION <<<
MAXIMUM STEEL RATIO =   .0282972
MAXIMUM STEEL AREA =     31.449 IN**2
MINIMUM GIRDER DEPTH =      95.614 INCHES
GIRDER TOP SLAB THICKNESS =      6.000 INCHES
GIRDER L+I DEFLECTION =   .6000000 INCHES
YC =  58.89720823 INCHES
CC =  40.90947970 INCHES

>>> CHECK FATIQUE REQUIREMENT <<<
LOGITUDINAL STEEL STRESS RANGE IS  12541.946 PSI.
ALLOWABLE FATIGUE STRESS RANGE IS  14540.164 PSI.
GIRDER FATIGUE REQUIEMENT OK.

>>> CHECK CRACK CONTROL CRITERIA <<<
LOGITUDINAL STEEL MAXIMUM STRESS IS  39389.935 PSI.
ALLOW. MAX. STRESS FOR CRACK CONTROL IS  30788.168 PSI.
GIRDER CRACK CONTROL NOT OK! INCREASE STEEL AREA.
```

```
>>> CHECK FATIQUE REQUIREMENT <<<
LOGITUDINAL STEEL STRESS RANGE IS  11385.978 PSI.
ALLOWABLE FATIGUE STRESS RANGE IS  15356.759 PSI.
GIRDER FATIGUE REQUIREMENT OK.

>>> CHECK CRACK CONTROL CRITERIA <<<
LOGITUDINAL STEEL MAXIMUM STRESS IS  35759.435 PSI.
ALLOW. MAX. STRESS FOR CRACK CONTROL IS  31951.070 PSI.
GIRDER CRACK CONTROL NOT OK! INCREASE STEEL AREA.

>>> CHECK FATIQUE REQUIREMENT <<<
LOGITUDINAL STEEL STRESS RANGE IS  10442.477 PSI.
ALLOWABLE FATIGUE STRESS RANGE IS  16023.263 PSI.
GIRDER FATIGUE REQUIREMENT OK.

>>> CHECK CRACK CONTROL CRITERIA <<<
LOGITUDINAL STEEL MAXIMUM STRESS IS  32796.225 PSI.
ALLOW. MAX. STRESS FOR CRACK CONTROL IS  33034.974 PSI.
GIRDER CRACK CONTROL OK.

>>> FINAL INVERT-T GIRDER DESIGN SOLUTION <<<
SPECIFIED GIRDER DEPTH =     102.000 INCHES
GIRDER TOP SLAB THICKNESS =       6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =       6.000 INCHES
GIRDER SECTION AREA =   2738.873 IN**2
GIRDER TOTAL VOLUME =   1901.995 FT**3
GIRDER DEAD LOAD DEFLECTION =  1.4041439 INCHES
GIRDER L+I DEFLECTION =    .5366791 INCHES
REQUIRED STEEL RATIO =    .0180506
REQUIRED STEEL AREA =      21.496 IN**2
PROVIDED STEEL RATIO =    .0223952
PROVIDED STEEL AREA =      26.670 IN**2
PROVIDE  21-# 10  IN  1 LAYERS

>>> COMPUTE SKIN REINFORCEMENT REQUIREMENT <<<
TOTAL SKIN REINFORCEMENT AREA =       7.421 IN**2
PROVIDE 19-#4 FRP BARS OVER THE CENTER WEB FACES
AT A DISTANCE (49.620 INCHES) ADJACENT TO THE MAIN REINFORCEMENT.

>>> DESIGN GIRDER SHEAR REINFORCEMENT <<<
SHEAR REINFORCEMENT DESIGN SOLUTION
-------------------------------------------------------------------------------
#5 STIRRUP,  SPACING =  14.95INCHES,  FROM    .00 FT.  TO  10.00 FT.  4 LEGS
#5 STIRRUP,  SPACING =  22.14INCHES,  FROM  10.00 FT.  TO  20.00 FT.  4 LES
#5 STIRRUP,  SPACING =  24.00INCHES,  FROM  20.00 FT.  TO  30.00 FT.  4 LEGS
#5 STIRRUP,  SPACING =  24.00INCHES,  FROM  30.00 FT.  TO  40.00 FT.  4 LEGS
#5 STIRRUP,  SPACING =  24.00INCHES,  FROM  40.00 FT.  TO  50.00 FT.  4 LEGS
-------------------------------------------------------------------------------

>>> GUIDEWAY SLAB REINFORCEMENT DESIGN <<<
USE #4 FRP REBAR AT  9.00 INCHES FOR POSITIVE AND NEGATIVE MOMENTS
```

5.2 Prestressed Concrete Girder Design Algorithm

1. Input design data:

   f_c' = specified compressive strength of concrete, in psi
   f_{pu} = specified tensile strength of prestressing strands, in psi
   f_{ci} = compressive strength of concrete at time of initial prestress
   W_D = girder dead load per ft
   W_{SD} = superimposed dead load per ft
   W_L = vehicle weight per ft
   L = girder span, in ft
   δ_a = allowable girder deflection, in inches

2. Input guideway section dimensions (Assume initial girder depth is 60 inches).

3. Compute allowable extreme fiber stresses at transfer and service:

   At transfer: extreme fiber stress in compression $f_{ci} = -0.6\ f_{ci}'$
   extreme fiber stress in tension $f_{ti} = 6\sqrt{f_{ci}'}$
   At service: extreme fiber stress in compression $f_c = -0.45 f_c'$
   extreme fiber stress in tension $f_t = 12\sqrt{f_c'}$

4. Compute initial strength of prestressing strand, f_{pi}. f_{pi} is assumed to be 70% of f_{pu}.

5. Find minimum section height that can provides the minimum required section modulus of the minimum efficient section for evaluating the concrete fiber stresses at the top and bottom fibers. this is done by using the secant method to find a height that satisfies

$$S_t = \frac{M_D + M_{SD} + M_L}{\gamma f_{ti} - f_c}$$

$$S_b = \frac{M_D + M_{SD} + M_L}{f_t - \gamma f_{ci}}$$

where S_t = section modulus for the top fibers
S_b = section modulus of the bottom fiber
M_D = moment due to girder self-weight
M_{SD} = moment due to superimposed dead load
M_L = moment due to live load, including impact
γ = residual prestress ratio

In the program, It is assumed that there is 20% prestress loss so r is equal to 0.8.

6. Use the secant method to find a minimum section height which have a live load plus impact deflection equal to the allowable deflection

7. Determine the minimum section depth. The minimum girder section depth is the least of sections heights from (5) and (6).

8. Enter the girder design depth. The chosen girder design depth should be greater than the minimum section depth so that the girder will have a live load plus impact deflection less than the allowable deflection.

9. Based on the chosen section, compute the prestressed force, eccentricity, and required prestressed area.

$$P_i = A_c f_{ci}$$

$$f_{cic} = f_{ti} - \frac{c_t}{h}(f_{ti} - f_{ci})$$

$$e_e = (f_{ti} - f_{cic})\frac{S_t}{P_i}$$

$$A_p = \frac{P_i}{f_{pi}}$$

where
- P_i = initial prestressing force
- A_c = girder section area
- f_{ci} = maximum allowable compressive stress in concrete immediately after transfer and prior to losses
- f_{cic} = concrete stress at transfer at the level of the centroid of the concrete section
- f_{ti} = maximum allowable tensile stress in concrete immediately after transfer and prior to losses
- c_t = the distance from the center of gravity of the section to the extreme top fiber
- h = girder section height
- e_e = the distance from the centroid of the girder section to the centroid of steel strands
- S_t = the section modulus for the top fiber
- A_p = prestressed steel area
- f_{pi} = the initial prestress

10. Select the steel strand size to be used for the girder.

11. Check for girder section fiber stress requirements.

(a) Analysis of stresses at transfer at support

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_e c_t}{r^2}\right) < f_{ti}$$

$$f^b = -\frac{P_i}{A_c}\left(1 - \frac{e_e c_b}{r^2}\right) > f_{ci}$$

(b) Analysis of final service-load stresses at support

$$f^t = -\frac{P_e}{A_c}\left(1 - \frac{e_e c_t}{r^2}\right) < f_t$$

$$f^b = -\frac{P_e}{A_c}\left(1 - \frac{e_e c_b}{r^2}\right) > f_c$$

(c) Analysis of final service-load stresses at midspan

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_e c_t}{r^2}\right) - \frac{M_t}{S_t} < f_c$$

$$f^b = -\frac{P_i}{A_c}\left(1 - \frac{e_e c_b}{r^2}\right) + \frac{M_t}{S_b} > f_t$$

where f^t = stress at the top fiber
f_b = stress at bottom fiber
r = radius of gyration of the gross girder section
f_t = maximum allowable tensile stress in concrete after losses at service load level
f_c = maximum allowable compressive stress in concrete after losses at service load level
M_t = Total moment due to gravity loads
S_b = the section modulus for the bottom fiber

If any of the stress requirements is not satisfied, a greater section depth has to be chosen. Thus, go to step 2.

12. perform ultimate-strength design of the prestressed girder

(a) calculate the required nominal moment capacity ($M_u/0.9$) from the factored loads.

(b) determine the steel stress at failure, f_{ps}. If $f_{pe} < 0.5\ f_{pu}$, strain compatibility analysis is required to find f_{ps}.

If $f_{pe} > 0.5f_{pu}$

$$f_{ps} = f_{pu}\left(1 - \frac{r_p}{\beta 1}\rho_p \frac{f_{pu}}{f_c'}\right)$$

(c) calculate the height of stress block.

$$a = \frac{A_{ps} f_{ps}}{0.85 f_c' b_w}$$

(d) find reinforcement index w_t

$$w_t = w_p = \rho_p \frac{f_{ps}}{f_c'}$$

(e) calculate the provided nominal moment capacity (M_n). If w_t is less than or equal to 0.36 β_1, the nominal strength is

$$M_n = A_{ps}\, f_{ps}\left(d_p - \frac{a}{2}\right)$$

If w_p is greater than $0.36\beta_1$, the section is overreinforced and the nominal strength is

$$M_n = f_c'\, b_w\, d_p^2\, (0.36\beta_1 - 0.08\, \beta_1^2)$$

(f) Check if the provided nominal moment capacity (M_n) is greater than the required nominal moment capacity ($M_u/0.9$).

5.2.1 Discussion on prestressed concrete girder design

1. The deflection calculation is based on the uncracked moment of inertia of the girder gross section. A more accurate calculation is to use the transformed section considering the prestressed steel area. However, it is found that the use of the moment of inertia of the gross section rather than the transformed section does not appreciably affect the accuracy sought in the calculations.

2. The deflection requirement (either 1/2000 or 1/4000) controls the girder section depth. The girder section depth satisfying the minimum section modulus is much less than that satisfying the deflection.

3. Although the section modulus for the girder with variable tendon eccentricity (harped steel strand) is less than that for the girder with constant tendon eccentricity (straight steel strand), the very small L+I deflection still controls the section design. Thus, the prestressed girder design in the program uses the straight steel strands (It is cheaper to construct).

4. Due to the greater section depth requirement for the very small deflection, the fiber stress requirements in the service-load design are much less than the allowable stresses.

5. At the present time, the program assumes that the loss of prestress is 20% of the initial prestressed force.

6. The overall cambered (arched upward) deflection due to the prestressed force, the dead load and the superimposed dead load is much higher than the L+I (downward) deflection.

7. Due to the very small deflection requirement, it is found that the provided nominal moment capacity is usually much greater than the required nominal moment capacity.

5.2.2 Output example #1

```
##########################
#  PROGRAM GWAY-PR  #
##########################



=============================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
=============================================================================


SECTION TYPE = BOX GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =        4500. PSI
TENSILE STRENGTH OF STEEL =   270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> BOX GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
WEB THICKNESS (TW) =    8.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
FLANGE LENGTH (FL) =  12.0 INCHES
FLANGE THICKNESS (TF) =  12.0 INCHES


=============================================================================
GIRDER DESIGN
=============================================================================


>>> FINAL PRESTRESSED BOX GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =      60.000 INCHES
GIRDER TOP SLAB THICKNESS =        6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =        6.000 INCHES
GIRDER SECTION AREA =   2141.977 IN**2
GIRDER TOTAL VOLUME =   1487.484 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) = -1.5822916 INCHES
GIRDER L+I DEFLECTION =   .5994930 INCHES
GIRDER SECTION CENTROID FROM TOP =      25.966 INCHES
STEEL STRAND ECCENTRICITY =      27.590 INCHES
PROVIDED STEEL STRAND AREA =      10.633 IN**2
PROVIDE  49 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =     88497121.913 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =    138778638.797 LB-IN
```

5.2.3 Output example #2

```
##########################
# PROGRAM GWAY-PR  #
##########################



==============================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = BOX GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =       4500. PSI
TENSILE STRENGTH OF STEEL =   270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .3 INCHES

>>> BOX GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
WEB THICKNESS (TW) =   8.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
FLANGE LENGTH (F ) =  12.0 INCHES
FLANGE THICKNESS (TF) =  12.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> FINAL PRESTRESSED BOX GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =      80.000 INCHES
GIRDER TOP SLAB THICKNESS =       6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =       6.000 INCHES
GIRDER SECTION AREA =   2460.227 IN**2
GIRDER TOTAL VOLUME =   1708.491 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) = -1.1822388 INCHES
GIRDER L+I DEFLECTION =   .2956276 INCHES
GIRDER SECTION CENTROID FROM TOP =      35.025 INCHES
STEEL STRAND ECCENTRICITY =      35.936 INCHES
PROVIDED STEEL STRAND AREA =      12.369 IN**2
PROVIDE  57 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =    96232373.276 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =   216595259.346 LB-IN
```

5.2.4 Output example #3

```
###########################
#  PROGRAM GWAY-PR  #
###########################



==========================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
==========================================================================


SECTION TYPE = DOUBLE-T GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =       4500. PSI
TENSILE STRENGTH OF STEEL =    270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> DOUBLE-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER WEB BOTTOM THICKNESS (BW) =  10.0 INCHES
WEB SPACING (BB) =  84.0 INCHES
TOP SLAB THICKNESS (TT) =   8.0 INCHES
FLANGE LENGTH (FL) =  12.0 INCHES
FLANGE THICKNESS (TF) =   8.0 INCHES


==========================================================================
GIRDER DESIGN
==========================================================================


>>> FINAL PRESTRESSED DOUBLE-T GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =      70.000 INCHES
GIRDER TOP SLAB THICKNESS =       8.000 INCHES
GIRDER SECTION AREA =   2324.000 IN**2
GIRDER TOTAL VOLUME =    1613.889 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) = -1.3429412 INCHES
GIRDER L+I DEFLECTION =   .5885419 INCHES
GIRDER SECTION CENTROID FROM TOP =      23.991 INCHES
STEEL STRAND ECCENTRICITY =      31.588 INCHES
PROVIDED STEEL STRAND AREA =       8.029 IN**2
PROVIDE  37 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =     92921298.320 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =   111831394.763 LB-IN
```

5.2.5 Output example #4

```
##########################
#  PROGRAM GWAY-PR  #
##########################



==============================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = DOUBLE-T GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =      4500. PSI
TENSILE STRENGTH OF STEEL =   270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .3 INCHES

>>> DOUBLE-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) = 120.0 INCHES
GIRDER WEB BOTTOM THICKNESS (BW) =  10.0 INCHES
WEB SPACING (BB) =  84.0 INCHES
TOP SLAB THICKNESS (TT) =   8.0 INCHES
FLANGE LENGTH (FL) =  12.0 INCHES
FLANGE THICKNESS (TF) =   8.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> FINAL PRESTRESSED DOUBLE-T GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =      90.000 INCHES
GIRDER TOP SLAB THICKNESS =        8.000 INCHES
GIRDER SECTION AREA =   2764.000 IN**2
GIRDER TOTAL VOLUME =   1919.444 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) = -1.0458102 INCHES
GIRDER L+I DEFLECTION =   .2910984 INCHES
GIRDER SECTION CENTROID FROM TOP =     32.560 INCHES
STEEL STRAND ECCENTRICITY =     38.337 INCHES
PROVIDED STEEL STRAND AREA =      10.416 IN**2
PROVIDE  48 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =   103615742.866 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =   184827585.001 LB-IN
```

5.2.6 Output example #5

```
##########################
#  PROGRAM GWAY-PR  #
##########################



==============================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = INVERT-T GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =       4500. PSI
TENSILE STRENGTH OF STEEL =    270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .6 INCHES

>>> INVERT-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) =  12.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
SIDE WEB THICKNESS (TW) =   6.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
TOP SLAB WIDTH (FL) =  96.0 INCHES
HEIGHT BETWEEN TOP SLAB AND GIRDER TOP (BW) =  42.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> FINAL PRESTRESSED INVERT-T GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =      84.000 INCHES
GIRDER TOP SLAB THICKNESS =       6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =       6.000 INCHES
GIRDER SECTION AREA =   2307.655 IN**2
GIRDER TOTAL VOLUME =   1602.538 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) = -1.1299394 INCHES
GIRDER L+I DEFLECTION =   .5575799 INCHES
GIRDER SECTION CENTROID FROM TOP =      53.341 INCHES
STEEL STRAND ECCENTRICITY =      11.795 INCHES
PROVIDED STEEL STRAND AREA =      19.096 IN**2
PROVIDE  88 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =    92524021.970 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =   481128988.75 LB-IN
```

5.2.7 Output example #6

```
##########################
#  PROGRAM GWAY-PR   #
##########################



==============================================================================
INPUT PRESTRESSED PRETENSIONING GIRDER DESIGN DATA
==============================================================================


SECTION TYPE = INVERT-T GIRDER
COMPRESSIVE STRENGTH OF CONCRETE =      6000.0 PSI
STRENGTH OF CONC. AT TIME OF INITIAL PRESTRESS =       4500. PSI
TENSILE STRENGTH OF STEEL =   270000.0 PSI
LIVE LOAD (WEIGHT OF MAG-LEV VEHICLE) = 1100.0 LB/FT
DEAD LOAD (WEIGHT OF CONCRETE) =  150.0 CU-FT
ADDITIONAL DEAD LOAD =  25.0 LB/FT
GIRDER SPAN = 100.0 FT
ALLOWABLE GIRDER DEFLECTION =    .3 INCHES

>>> INVERT-T GIRDER SECTION DIMENSIONS <<<
GIRDER TOP WIDTH (B) =  12.0 INCHES
GIRDER BOTTOM WIDTH (BB) =  84.0 INCHES
SIDE WEB THICKNESS (TW) =   6.0 INCHES
TOP SLAB THICKNESS (TT) =   6.0 INCHES
BOTTOM SLAB THICKNESS (TB) =   6.0 INCHES
TOP SLAB WIDTH (FL) =  96.0 INCHES
HEIGHT BETWEEN TOP SLAB AND GIRDER TOP (BW) =  42.0 INCHES


==============================================================================
GIRDER DESIGN
==============================================================================


>>> FINAL PRESTRESSED INVERT-T GIRDER DESIGN <<<
SPECIFIED GIRDER DEPTH =     104.000 INCHES
GIRDER TOP SLAB THICKNESS =        6.000 INCHES
GIRDER BOTTOM SLAB THICKNESS =        6.000 INCHES
GIRDER SECTION AREA =   2786.803 IN**2
GIRDER TOTAL VOLUME =   1935.280 FT**3
GIRDER DEAD LOAD DEFLECTION (CAMBER) =  -.9126520 INCHES
GIRDER L+I DEFLECTION =   .2856112 INCHES
GIRDER SECTION CENTROID FROM TOP =       62.940 INCHES
STEEL STRAND ECCENTRICITY =      16.367 INCHES
PROVIDED STEEL STRAND AREA =     21.700 IN**2
PROVIDE 100 - .600 IN. NOMINAL DIAMETER STEEL STRANDS
REQUIRED NOMINAL MOMENT CAPACITY =   104169983.084 LB-IN
PROVIDED NOMINAL MOMENT CAPACITY =   713238443.27 LB-IN
```

5.3 Column Design

5.3.1 Rectangular column design algorithm

1. Input design data:

P_u	=	factored axial load
M_{ux}	=	factored moment about x axis
M_{uy}	=	factored moment about y axis
CH	=	dimension on the long side
RH	=	ratio of distance between centroids of outer rows of bars and thickness of cross section, in the direction of bending about the x axis
CB	=	dimension on the short side
RB	=	ratio of distance between centroids of outer rows of bars and thickness of cross section, in the direction of bending about the y axis

2. Use the minimum steel ratio (ρ_g = 1%) and maximum steel ratio (ρ_g = 8%) to check the column size based on the input axial load and moments. If less than 1 % steel ratio is needed, the column size can be reduced, and if more than 8 % steel ratio is required, then the column size has to be increased.

3. Use the secant method to find the optimum steel ratio. The initial two trial solutions are ρ_g = 1% and ρ_g = 8%. For a given steel ratio, the program computes

   (a) the P_{nx} and M_{nx} (nominal axial load and moment strength for eccentricity $e_y = M_{ux}/P_u$ along the y-axis only, x-axis is the axis of bending), and the P_{ny} and M_{ny} (nominal axial load and moment strength for eccentricity $e_x = M_{uy}/P_u$ along x-axis, y is the axis of bending)

   (b) the P_o (nominal axial load strength for zero eccentricity)

   (c) the P_{nxy} (approximation of nominal axial load strength at eccentricities e_x and e_y) using Equation 19

   (d) the ratio RM

$$RM = \frac{P_u}{\phi P_{nxy}}$$

   when the factored load,

$$P_u \geq \text{to } 0.1 f_c' A_g,$$

or

$$RM = \frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \qquad \text{using}$$

Equation 21

when the factored load,

$$P_u < 0.1 f_c' A_g.$$

The solution converges when a steel ratio gives the RM value equal to 1.

4. Determine the steel arrangement.

5.3.2 Round column design algorithm

1. Input design data

P_u	=	factored axial load
M_u	=	factored moment (equal to $\sqrt{M_{ux}^2 + M_{uy}^2}$)
CH	=	diameter
R	=	ratio of distance between centroids of outer rows of bars and thickness of cross section, in the direction of bending

2. Use the minimum steel ratio (ρ_g = 1%) and maximum steel ratio (ρ_g = 8%) to check the column size based on the input axial load and moments. If less than 1% steel ratio is needed, the column size can be reduced, and if more than 8% steel ratio is required, then the column size has to be increased.

3. Use the secant method to find the optimum steel ratio. The initial two trial solution are ρ_g = 1% and ρ_g = 8%. For a given steel ratio, the program computes

   (a) the P_n and M_n (nominal axial load and moment strength for eccentricity $e = M_u/P_u$)

   (b) the ratio RM

$$RM = \frac{P_u}{\phi P_{nx}}$$

The solution converges when a steel ratio gives the RM value equal to 1.

4. Determine the steel arrangement.

5.3.3 Discussion on column design

1. In the rectangular column design, four different column reinforcement arrangements are considered:

   (a) total steel area is equally distributed on four sides
   (b) total steel area is proportionally distributed on four sides
   (c) total steel area is equally distributed on two short sides
   (d) total steel area is equally distributed on two long sides

2. The rectangular column were verified by examples 14 and 15 in the ACI Design Handbook Volume 2 - Columns (ACI 340.2R-90).

3. For round column design, both regular and spiral ties are considered (ϕ = 0.7 for tied column and ϕ = 0.75 for spiral column)

4. The round column were verified by example 7 in the ACI Design Handbook Volume 2 - Columns (ACI 340.2R-90) and example 10.6 in Notes on ACI318-83 by Portland Cement Association.

5. The column design program will be combined with the the reinforced guideway girder design program and the prestressed guideway girder design program. Thus, the column design data for the factored axial load and moments can be generated automatically.

5.3.4 Output example #1

```
######################
#  PROGRAM BICOLUMN  #
######################



>>> RECTANGULAR COLUMN DESIGN DATA <<<


CONCRETE COMPRESSIVE STRENGTH =   5000.000 PSI
STEEL STRENGTH =  60000.000 PSI
DIMENSION PARALLEL TO X AXIS =     15.000 INCHES
DIMENSION PARALLEL TO Y AXIS =     15.000 INCHES
R RATIO IN X DIRECTION =       .670
R RATIO IN Y DIRECTION =       .670
FACTORED AXIAL LOAD =      208000.000 LB
FACTORED MOMENT ABOUT X AXIS =     2064000.000 LB-IN
FACTORED MOMENT ABOUT Y AXIS =      828000.000 LB-IN


>>> RECTANGULAR COLUMN REINFORCEMENT DESIGN <<<


STEEL RATIO =   .0434442
REQUIRED STEEL AREA =      9.775
PROVIDED STEEL AREA =     10.160
USE  8 #10 ( 3 IN X-X FACE,  3 IN Y-Y FACE)
```

5.3.5 Output example #2

```
######################
#  PROGRAM BICOLUMN  #
######################



>>> RECTANGULAR COLUMN DESIGN DATA <<<


CONCRETE COMPRESSIVE STRENGTH =   5000.000 PSI
STEEL STRENGTH =  60000.000 PSI
DIMENSION PARALLEL TO X AXIS =     12.000 INCHES
DIMENSION PARALLEL TO Y AXIS =     18.000 INCHES
R RATIO IN X DIRECTION =      .580
R RATIO IN Y DIRECTION =      .720
FACTORED AXIAL LOAD =      208000.000 LB
FACTORED MOMENT ABOUT X AXIS =     2064000.000 LB-IN
FACTORED MOMENT ABOUT Y AXIS =      828000.000 LB-IN


>>> RECTANGULAR COLUMN REINFORCEMENT DESIGN <<<


STEEL RATIO =   .0425716
REQUIRED STEEL AREA =      9.195
PROVIDED STEEL AREA =     10.160
USE  8 #10 ( 3 IN X-X FACE,  3 IN Y-Y FACE)
```

5.3.6 Output example #3

```
######################
#  PROGRAM BICOLUMN  #
######################



>>> ROUND SPIRAL COLUMN DESIGN DATA <<<


CONCRETE COMPRESSIVE STRENGTH =   5000.000 PSI
STEEL STRENGTH =  60000.000 PSI
DIAMETER =      17.000 INCHES
R RATIO =        .670
FACTORED AXIAL LOAD =      940000.000 LB
FACTORED MOMENT ABOUT X AXIS =      480000.000 LB-IN


>>> ROUND COLUMN REINFORCEMENT DESIGN <<<


STEEL RATIO =    .0402907
REQUIRED STEEL AREA =      9.145
PROVIDED STEEL AREA =      9.360
USE  6 #11
```

5.3.7 Output example #4

```
######################
#  PROGRAM BICOLUMN  #
######################



>>> ROUND SPIRAL COLUMN DESIGN DATA <<<


CONCRETE COMPRESSIVE STRENGTH =    5000.000 PSI
STEEL STRENGTH =  60000.000 PSI
DIAMETER =      16.000 INCHES
R RATIO =        .670
FACTORED AXIAL LOAD =       236000.000 LB
FACTORED MOMENT ABOUT X AXIS =     2148000.000 LB-IN


>>> ROUND COLUMN REINFORCEMENT DESIGN <<<


STEEL RATIO =    .0503870
REQUIRED STEEL AREA =     10.131
PROVIDED STEEL AREA =     10.920
USE  7 #11
```

5.4 Spread Footing Design Algorithm

1. Input design data:

P = service axial load
M_x = service moment about x axis
M_y = service moment about y axis
P_u = factored axial load
M_{ux} = factored moment about x axis
M_{uy} = factored moment about y axis
FH = dimension on the long side
FB = dimension on the short side
FT = thickness of the spread footing
FG = distance from the ground level to the bottom of the footing
Q_a = allowable soil pressure

2. Use service loads to calculate the maximum and minimum pressures under the footing

3. Check if the minimum pressure is negative. If yes, increase the footing size until the minimum pressure is greater or equal zero.

4 Check the maximum pressure with the allowable soil pressure. If the maximum pressure is greater than the allowable soil pressure, change the allowable soil pressure or increase the footing size

5. Calculate the factored soil pressures under the footing due to the factored loads acting on the footing.

6. Based on the calculated footing pressures, the factored moments per foot are computed by cantilever action at the corner of the column shaft in the transverse as well as longitudinal directions.

7. Use the strength design procedure for a singly reinforced concrete beam is then applied to the two directions to find the reinforcement for the footing.

5.5 Guideway Structural Design Output Examples

Design examples use GWAY0 and GWAY1 will be given in the final report.

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