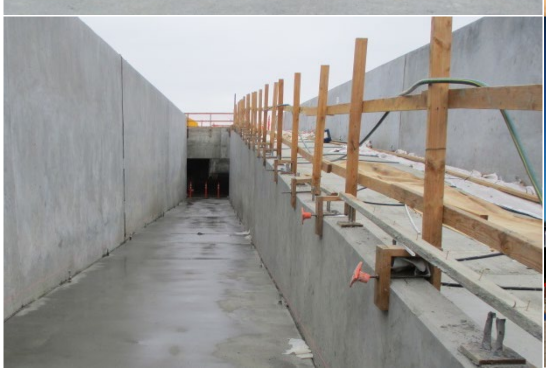
PENINSULA CORRIDOR JOINT POWERS BOARD

STANDARDS FOR DESIGN AND MAINTENANCE OF STRUCTURES








REVISED January 1, 2024

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INTRODUCTION

Purpose

The Peninsula Corridor Joint Powers Board (PCJPB) has developed and has been implementing a Capital Improvement Program for infrastructure improvements since its inception. Capital improvements include replacement or rehabilitation of existing structures and railroad bridges on the corridor. Additionally, there are plans to eliminate at-grade railroad crossings at various locations on the corridor by constructing grade separation structures among other projects involving structures.

The Standards for Design and Maintenance of Structures are based on existing guidelines, regulations, specifications, codes, documents, best industry practices, and recommended issues relevant to the operations of the PCJPB facilities. The intent is to provide project design guidelines to help guide PCJPB designers and consultants working on projects related to PCJPB facilities in providing uniformity in design along the PCJPB corridor.

The PCJPB operations will continue to expand with an increase in rail traffic along this corridor. Electrification of the railroad is currently under construction and the State of California is proposing high-speed rail along the corridor for the future. On a typical weekday, currently there are 92 trains operating on the corridor with plans to increase the service level further over time. Due to the volume of rail traffic and the desire to maintain passenger operations schedules, it is anticipated that only limited work windows (mostly nights and weekends) will be allowed to construct new facilities where the main line tracks are anticipated to be encroached upon by construction activities.

The PCJPB operations and their contract operator will provide passenger service while freight rail traffic will continue to be operated along the corridor.

The guidelines for each structure type have been developed to address preferred design types, types of loads, load combinations, load application, structure material strength and properties, and define requirements for structure type selection.

Roadway Worker Protection Training (RWP Training)

Working near Caltrain tracks can be dangerous and the PCJPB's first priority is for safety along their corridor. All Consultants, Designers (including surveyors, geotechnical, and other sub-consultants), Contractors, and employees that will be performing services that require them to enter PCJPB operating right-of-way are required to be trained and currently certified in the PCJPB Roadway Worker Protection Training program before entering the property. Details regarding training and certification are available from the PCJPB. The adoption of this safety procedure is prescribed by the Federal Railroad Administration (FRA) in 49 CFR 214 – Railroad Workplace Safety.

Qualification Clause

The information prepared herein is presented as a general guide and will be used under the direction of PCJPB by qualified and experienced engineering consultants or staff. In that regard the PCJPB requires that qualifications of the lead designer or designer in responsible charge of railroad bridge projects have performed 5 years of qualifying railroad bridge design and construction, have a current membership in the American Railway Engineering and Maintenance-of-Way Association (AREMA), and preferably be involved current AREMA committees.

The guidelines are not intended as a substitute for, formal design criteria code application, or the professional experience and judgement of the design professional performing specific services for PCJPB. Each consultant's use of the guidelines will be tailored to the unique tasks and circumstances associated with a particular site or project.

Data presented in the guidelines is representative and is not intended to be exhaustive, precise, or useful for every application. By using the guidelines the user assumes all responsibility for its use. PCJPB and the contributors to this document do not assume or accept any responsibility or liability, including liability for negligence, for errors or oversight, for the use of the guidelines in preparing engineering plans or designs.

CHAPTER 1

GENERAL

CHAPTER 1: GENERAL INFORMATION AND BACKGROUND

1.1 Application of Guidelines

These guidelines and procedures provide for a uniform basis for the structural design criteria for the Peninsula Corridor Joint Powers Board (PCJPB) – Caltrain projects. Structure items included in this guide include railroad bridges and underpasses, pedestrian underpasses and overhead, retaining walls, and other miscellaneous structures. Where there are cases of special design types, special projects or large- scale projects encountered that are not specifically covered by these guidelines, the designer shall bring them to the attention of PCJPB to determine the applicable criteria to use.

These guidelines also provide some rules, regulations and standards of practice for all personnel performing design, maintenance, and construction on PCJPB owned or maintained track and closely related facilities. The manual is intended to be used and applied by employees of the PCJPB staff, the Contract Operator (Amtrak) as well as construction contractors, PCJPB engineering consultants, and any other parties involved in supervising or directly performing design, construction, maintenance or inspection of track owned or maintained by the PCJPB.

The Union Pacific Railroad (UPRR) also operates within the PCJPB corridor and in certain instances the guidelines and standards of the UPRR will govern the design. The designer shall discuss with the PCJPB where and if the UPRR requirements will govern.

The design of a structure owned or maintained by an agency other than the PCJPB shall be in accordance with the standards used by that agency, except where the following conditions exist. If the structure encroaches onto the PCJPB right-of-way or the structure could potentially impact or pose a restriction to the PCJPB’s full use of its right-of-way for operating purposes the PCJPB shall be contacted for the guidelines that apply.

These design guidelines shall be used in conjunction with other applicable governmental rules and regulations and manuals of practice. If these guidelines conflict with the application of another guideline or code, the conflict shall be brought to the attention of the PCJPB for resolution.

These guidelines and procedures serve as recommended practice and do not substitute for engineering judgement and sound engineering practice. Unless specifically noted otherwise in these guidelines and procedures, the latest edition of guidelines, codes, regulations, and standards that are applicable at the time the design is initiated shall be used. If a new addition or an amendment to a guideline, code, regulation, or standard is issued before the design is completed, the design shall conform to the new requirements to the extent presented or required by the agency enforcing the requirement. Specific exceptions may apply in special cases.

Any variances to the guidelines and procedures must be approved by PCJPB prior to use in the design. Application for change of guidelines and procedures and other questions should be submitted in writing with full details pertaining to the issues to:

Deputy Director of Engineering–
Caltrain
1250 San Carlos Ave. – P.O. Box 3006
San Carlos, CA 94070-1306

1.2 Compatibility

These guidelines shall be used in a way that they are compatible with the other PCJPB guidelines, standards, and operations in effect.

These PCJPB documents and operations include:

- PCJPB Engineering Standards for Excavation Support Systems
- Caltrain Engineering Standards
- Standard Procedures for Track Maintenance and Construction
- Caltrain Timetable
- PCJPB CADD Standards

1.3 Design Guidelines, Codes, Manuals, Standards, and Specifications

The primary structural design guideline to be used in the design of PCJPB structures shall be the American Railway Engineering and Maintenance-of-Way Association (AREMA) – Manual of Railway Engineering (latest edition) as may be modified by these guidelines. All designers doing work that is related to PCJPB structures, operations, or right-of way shall have the latest edition of this document.

Other governing guidelines, codes, manuals, and specifications are listed below:

- Federal Railroad Administration (FRA) 49 CFR 213 – Track Safety Standards
- FRA 49 CFR 237 – Bridge Safety Standards
- California Public Utilities Commission (CPUC) General Orders
- AASHTO Load and Resistance Factor Design Bridge Design Specifications with California Amendments

Note: The Caltrans Bridge Design Details Manual, the Caltrans Memo to Designers Manual, and other Caltrans Manuals that have sections on railroad structure design shall have their use verified with the PCJPB staff prior to incorporating those provisions in the design. A determination will be made on which sections are applicable to the PCJPB project.

1.4 General Guidelines for Grade Separations

The PCJPB prefers overhead highway crossings to underpasses where practical. When overhead highway crossings or underpass highway crossings are planned it is preferable to not change the alignment or elevation of the existing main tracks. Considerations for track alignment and grade changes shall be discussed with the PCJPB for input on the impact to rail operations and the impacts during the construction phase. The designer shall make provisions in designing new facilities to not preclude future four-track expansion, when feasible.

Agencies intent on designing or constructing grade separations passing over or under the tracks and/or property of the PCJPB should consult with the Deputy Director of Engineering early in the planning stage to verify the design requirements given herein and to determine the requirements regarding train operations within the corridor. Construction affecting train service should be avoided or minimized. If impacts to operations are unavoidable, impact time and duration must be approved by PCJPB and the project design and construction sequencing must be approved by PCJPB. The following listing provides some of the issues that the PCJPB will be concerned with during design:

- Design requirements for structures and including track alignment and grades
- Permanent impacts to train operations
- Construction impacts to train operations (temporary)
- Construction staging and sequencing

The requirements addressed in this guideline should be followed for all structures over or under the PCJPB tracks or structures constructed on PCJPB property. Compliance with the requirements herein will expedite the review and approval of project submittals.

1.5 Distribution

The PCJPB Standards for Design and Maintenance of Structures manual is distributed to PCJPB staff, contract operator staff, consultants, and contractors whose responsibilities include design, construction, and maintenance of the PCJPB track, structures, and right of way.

1.6 Applicable Federal Rules, Regulations, Instructions and Standards Related to Maintenance and Construction on the Railroad

1.6.1 Federal Railroad Administration Regulations

The following federal regulations are specifically related to maintenance of way:

FRA Standards
Part 213 – Track Safety Standards
Subpart A – General
Subpart B – Roadbed Subpart
Subpart C – Track Geometry
Subpart D – Track Structure
Subpart E – Track Appliances and Track-Related Devices
Subpart F – Inspection
Appendix A – Maximum allowable Curving Speeds
Appendix B – Schedule of Civil Penalties
Appendix C – Statement of Agency Policy on Safety of Railroad Bridges
Defect Codes for Subparts A-F
Part 214 Railroad Workplace Safety
Subpart A – General

FRA Standards
Subpart B – Bridge Worker Safety Standards
Subpart C – Roadway Worker Protection
Part 217 – Railroad Operating Rules
Part 219 – Control of Alcohol and Drug Use
Part 220 – Railroad Communications
Part 225 – Railroad Accidents/Incidents: Reports Classification and Investigations
Part 228 – Hours of Service for Railroad Employees
Part 231 – Railroad Safety Appliance Standards
Part 233 – Signal Systems Reporting Requirements
Part 234 – Grade Crossing Signal System Safety
Part 235 – Applications to Discontinue a Signal System
Part 236 – Rules and Standards Governing Installation and Repair of Signal and Train Control Systems
Part 237 – Bridge Safety Standards

1.6.2 Occupational Safety and Health Administration (OSHA) Part 1910 Gen Industry

This section includes the confined space regulations, Part 1910.146.

1.6.3 Occupational Safety and Health Administration (OSHA) Part 1926 Construction

- Subpart E – Personal Protective Equipment
- Subpart K – Welding and Cutting
- Subpart M – Fall Protection
- Subpart N – Cranes, Derricks and Hoists
- Subpart O – Motor Vehicles and Mechanized Equipment
- Subpart P – Excavations
- Subpart X – Ladders
- Subpart Z – T
- Toxic and Hazardous Substances

1.7 Applicable State Rules, Regulations, Instructions and Standards Related to Maintenance and Construction on the Railroad

1.7.1 California Public Utilities Commission (CPUC) General Orders (GO)

- 22-8 Report of Accidents on Railroads
- 26-D Clearance on Railroads and Streets
- 72-B Pavement at Railroad Crossings
- 75-C Signs and Warning Devices Protection at RR Grade Crossings
- 88-A Altering Public Railroad Highway Crossings

- 96-A Electrical Transmission
- 118 Walkways Adjacent to Tracks
- 135 Blocking at RR Grade Crossings

1.8 Other Standards and References

1.8.1 Manual of Uniform Traffic Control Devices (MUTCD)

This manual provides standards for traffic control, including striping, signage, and traffic signals.

CHAPTER 2

RAILROAD BRIDGES AND UNDERPASSES

CHAPTER 2: RAILROAD BRIDGES AND UNDERPASSES

Purpose

This guideline was created to provide standards and requirements regarding design and construction of new or modified existing grade separation underpass structures that affect the tracks and property of the Peninsula Corridor Joint Powers Board (PCJPB). Design engineers should use these guidelines as the basis for preliminary and final design.

2.1 General Requirements

The primary structural design guidelines to be used in the design of PCJPB structures shall be the American Railway Engineering and Maintenance-of-Way Association (AREMA) – Manual of Railway Engineering (latest edition) as may be modified by these guidelines.

If the detailed requirements are not specifically covered in AREMA then the appropriate section of the current edition of Caltrans Bridge Design Specifications shall be applied. Final determination of the appropriate sections of Caltrans BDS shall be discussed with the PCJPB.

Right of Entry, Right-of-Way encroachment, and Construction and Maintenance (C&M) agreements consistent with PCJPB policy may be required depending on the type of project, who is the lead agency, and to establish the responsibilities of all parties.

The municipality, other public agency, or PCJPB will be required to make application to the California Public Utilities Commission (CPUC) for a new or any modification to a grade separation. The PCJPB staff will support the application, if required, provided that it concurs with their long range planning. If the lead agency is the PCJPB then the designer will be required to provide the exhibit material necessary to accompany the PUC application process.

For submittal requirements, see Chapter 7.

The agency or their representative shall provide the basic information requested on the Underpass Project Sheet to PCJPB at the initial stage of the project. A copy of the Underpass Project Sheet is included in Chapter 7.

2.2 Superstructure Selection Type

Structure type selection shall be discussed with the PCJPB to determine and identify any constraints that may control the design that are not listed in this guideline. Type selection shall at least be based on the following factors to determine the most suitable type of structure for the location:

- Location
- Configuration
- Cost
- Maintainability
- Constructability
- Impacts on Rail Operations
- Aesthetic Considerations

- Environmental

Grade separation underpass structures built to support railroad tracks shall be ballast deck type structures. Railroad timber open deck type structures shall only be considered for temporary structures or shoofly structures. Railroad direct fixation type structures shall not be considered for the PCJPB corridor for permanent or temporary structures.

Provisions shall be discussed with PCJPB on the need to accommodate additional or future main tracks across the structure for each project. A four track alignment shall be considered as it is part of the future planning for the corridor.

In the design of underpasses the top of rail elevation and alignment are preferred to remain at the existing location. Where it is required that the track grades or alignment be changed do to the circumstances of design, the PCJPB requests that it be consulted and the following considerations be given:

- The change in track grade for both temporary and permanent alignment shall minimize the impact on train operations, adjacent station platforms, parking lots, and railway maintenance access.
- The change in track grade shall minimize the undulating effect on the track profile relative to existing underpasses, future underpasses, or nearby grade crossings.
- The change in track grade that will require adjustments or relocations to the fiber optic lines along the corridor shall be identified and solutions investigated.

During the design process all property and right-of-way requirements shall be identified that will be required for the project. Permanent right-of-way requirements and temporary construction easements shall be brought to the attention of the PCJPB as early as possible.

Simple span structures are preferred over a continuous span type of superstructure for use along the corridor. The use of continuous type superstructures will only be considered when conditions preclude any other solution. Continuous type structures require the approval of the PCJPB Deputy Director of Engineering.

Deck type structures are preferred over through type structures. The concrete through trough type of prestressed or post tensioned, simple or continuous structures, are not to be considered.

2.2.1 List of Preferred Structure Types

The following list of superstructure types is listed in priority order as the preferred types for use along the PCJPB Corridor: (1 being the most preferred and 8 being the least preferred)

1. Steel rolled beams (four or more beams per track) with a cast-in-place concrete or steel deck. (See Figure 2.7 at the end of this chapter)
2. Steel plate girders (four or more girders per track) with a cast-in-place concrete or steel deck. (See Figure 2.7 at the end of this chapter)
3. Prestressed concrete box girders (single or double cell) or solid slab girders (no void). (See Figure 2.8 at the end of this chapter)

4. Steel rolled beams or plate girders (two beams/girders per track) with a cast-in-place concrete deck. (See Figure 2.9 at the end of this chapter)

5. Prestressed concrete “AASHTO” type girders with cast-in-place diaphragms and deck. (See Figure 2.10 at the end of this chapter)

6. Cast-in-place concrete box girders, conventionally reinforced. (See Figure 2.11 at the end of this chapter)

7. Post tensioned concrete box girders. (See Figure 2.11 at the end of this chapter)

8. Through type steel structures (girder or truss) with a cast-in-place concrete or steel deck. Although low profile (top flange near the top of rail in height) through plate girders are more desirable than deep through plate girders. (See Figure 2.12 at the end of this chapter)

See the end of this chapter for typical sections (Figure 2.7 through 2.12) of these structure types and other preliminary planning information related to each type. The span depth factor given in the figures is for simple span structures and is provided as a preliminary layout aid in engineering. Final superstructure depth shall be based on AREMA requirements for stress and deflection. The comparative cost ranking is based on the highest cost being a 10 and the lowest cost being a 1.

2.3 Structural Design Requirements

2.3.1 Layout

The project plans involving an underpass structure shall indicate the limits of PCJPB right-of-way and any other public or private right-of-way involved. Locations of all existing and proposed underground and overhead utilities shall be located and indicated. Proposed construction staging and requirements for a temporary bridge structure or shoofly arrangement and design shall be established. All proposed construction shall be designed and staged to minimize the amount of track interference during construction.

Utility lines will not be permitted to be attached to the bridge superstructure unless approved by the PCJPB, except for utilities (i.e., railroad signal and communication lines) that are required for the operation of the railroad. Signal cables may be buried in the ballast across a bridge, but away from the track. Where railroad signal and communication lines are to be attached to the superstructure special mounting provisions (exterior brackets) shall be designed that minimize interference with maintenance and inspection activities of the structure. Existing or future fiber optic lines shall be placed underground and away from the bridge structure. In most cases relocation of existing utilities will be by the owners of the utility. In some instances where the utility line has no other option than to be located on the structure; the PCJPB will handle this situation on a case by case basis. In the case where a utility line is allowed on a structure it may only be attached to the substructure of the bridge and the following requirements apply:

- Utility line must have a minimum separation of 2 feet horizontality from the superstructure
- Utility cannot be attached to the superstructure
- Utility can be attached to the pier and abutment caps, if they are extended for that purpose
- There are no penetrations of the abutment, backwall, or wingwall for the utility line.

Other utility lines that are impacted by the bridge construction shall be identified. Impacted utility lines may require relocation, protection, encasement, or a casing.

Bridge deck drainage shall be provided for and directed away from the track roadbed and away from the back of bridge abutments. Bridge deck shall have a minimum of 0.4% grade longitudinally. Natural drainage that occurs toward the bridge ends shall not be allowed to drain onto the bridge deck but shall be intercepted prior to it draining onto a bridge. Drainage patterns for the entire right-of-way shall be considered for the project with all drainage directed away from the tracks.

For a bridge located on a curve alignment, the girders, abutments, and piers shall be positioned with reference to chords.

The year the bridge was constructed shall be embedded into the face of the concrete backwall where it is readable. Letters shall be 6 inches tall. The bridge milepost designation shall also be shown on all the design plans. Contact the PCJPB if you need help in establishing the PCJPB bridge milepost for your project.

Structures shall be designed to accommodate any future shifting or relocation of track within the limits of the bridge deck. Longitudinal members are to be evenly spaced.

Abutment stem walls shall be at least 0.2h in thickness at their base. Columns and piers shall be at least 0.2h in thickness at the base per Figure 2.1. Minimum bridge seat width shall be per AREMA requirements and shall follow the requirements of these standards per Section 2.6.1. All Abutments shall incorporate a back wall to prevent ballast and the abutment backfill material from migrating to the bridge seat.

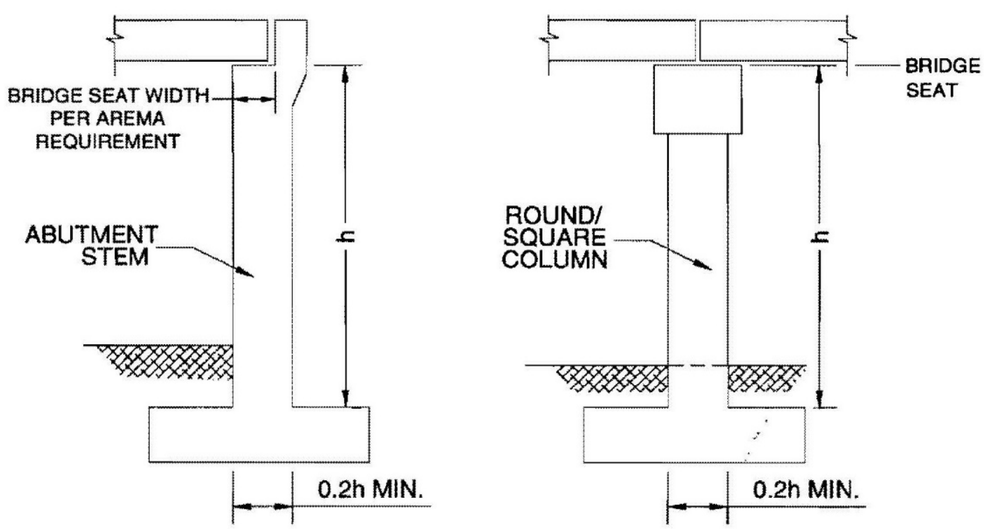


FIGURE 2.1 – ABUTMENT AND COLUMN REQUIREMENTS

Main line track centers for new bridge construction shall be a minimum of 15’. Track centers shall be verified with PCJPB staff prior to design, as there are other track arrangements that need to be considered (such as at station locations, etc.).

Consideration shall be given in the design of the superstructure, abutments, piers, and bridge seats to allow for the future addition of a third and fourth main tracks along the PCJPB corridor. Similarly, the profile of the roadway under the track shall provide the vertical clearance necessary for the future tracks. Requirements for future tracks shall be determined prior to beginning design.

The designer should be aware that the track structure is supported by the bridge structure, the combination of track and bridge movement (deflection) cannot exceed the tolerances in track standards. Refer to Federal Railroad Administration (FRA) 49 CFR – Part 213 Track Safety Standards for requirements on track tolerances. Therefor it is important for the designer to not exceed the deflection requirements established in the AREMA recommendations.

The use of metal inner guard rails is not generally required on the track by the PCJPB. But in some situations the PCJPB may decide that it is prudent to provide guard rails at locations where track alignment, train speed, train density, type of train traffic, and type of structure is such that this additional precautionary measure is warranted. Typically the types of structures that will be considered for guard rails are:

a) On tracks of through-span bridges.
b) On tracks of open floor deck bridges.
c) On tracks where the centerline is within 10 feet of the support of an overhead or an adjacent structure.
d) On tracks through tunnels.

2.3.2 Bridge Skew

The preferred angle of crossing and bridge structure relative to the centerline of track is 90 degrees. In cases where a 90-degree crossing cannot be obtained the maximum skew of a bridge from the 90-degree shall not exceed 30 degrees. Prior approval from the PCJPB shall be obtained for any skew where a 90-degree crossing cannot be obtained. Skews in excess of 15 degrees are not allowed for continuous span structures and require approval of the PCJPB for the skew and the continuous structure.

If concrete structures use transverse tie rods in the end blocks and at interior diaphragms the tie rods are preferred to be perpendicular to the girder, although in some cases upon approval of the PCJPB skewed tie rods may be used. Multiple single cell concrete girders over 45 feet shall be bonded together with an epoxy grout. For steel girder span bridges the transverse diaphragms shall be perpendicular to the girders but staggered across the width of the structure in line with the skew and at the ends of the girder near the bearings a skewed diaphragm may be allowed.

If skewed abutments are approved, support perpendicular to the track alignment at the end of the skewed structure shall be provided. This squaring off of the support shall be an integral part of the abutment as shown in Figure 2.2.

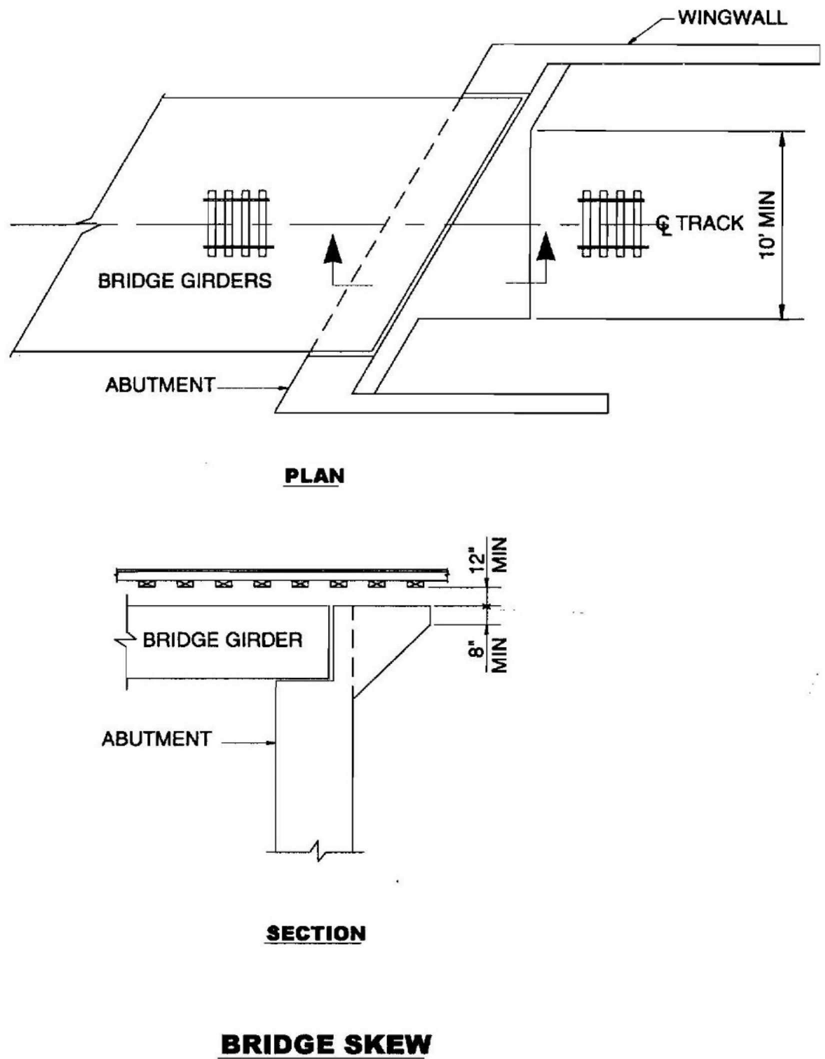


FIGURE 2.2 – TRACK PERPENDICULAR TO SUPPORTING STRUCTURE

Each bridge shall have a track transition at the location where the track comes onto the structure whether the bridge is skewed or not. This transition shall be a 12-inch hot mix asphalt concrete (HMAC) underlayment at each bridge approach extending 50 feet from the bridge abutments. There shall be 9 inches of ballast between the HMAC and the bottom of the tie. Across the bridge section there shall be 4-inches of HMAC on the bridge deck and the ballast depth shall remain at 9 inches beneath the tie. In addition the track ties for this transition area shall be 10 feet long. See Figure 2.3.

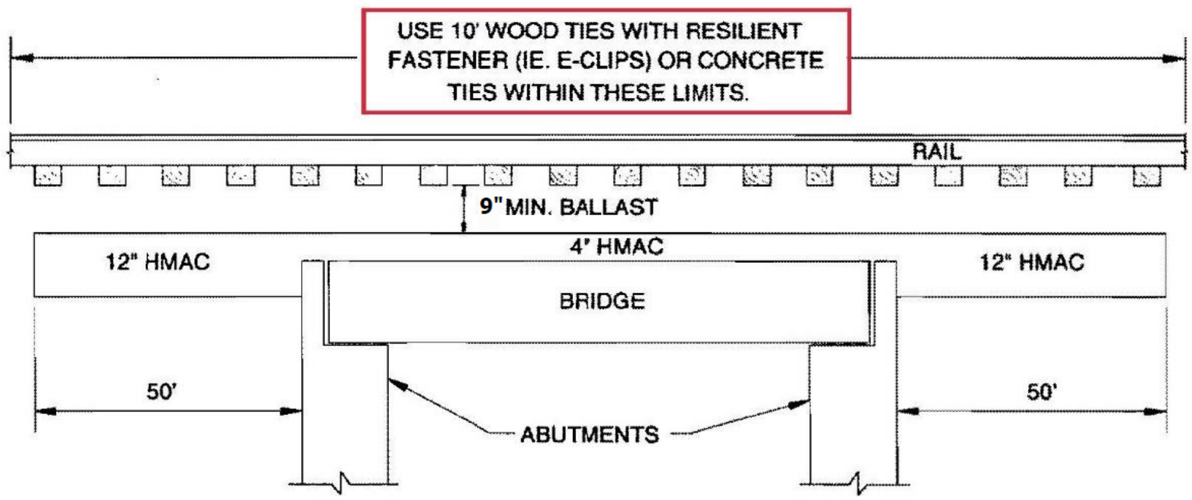


FIGURE 2.3 – TRACK TRANSITION AT BRIDGE

2.3.3 Design Loads for Railroad Bridge Structures

Underpass bridge structures shall be designed for the loads and loading conditions specified in the appropriate chapter, for the material being considered, of AREMA – Manual tor Railway Engineering. For further guidance on the application of loads and their distribution to the structure refer to the Design Sections in AREMA and to its commentary as well as to the Commentary given in these standards.

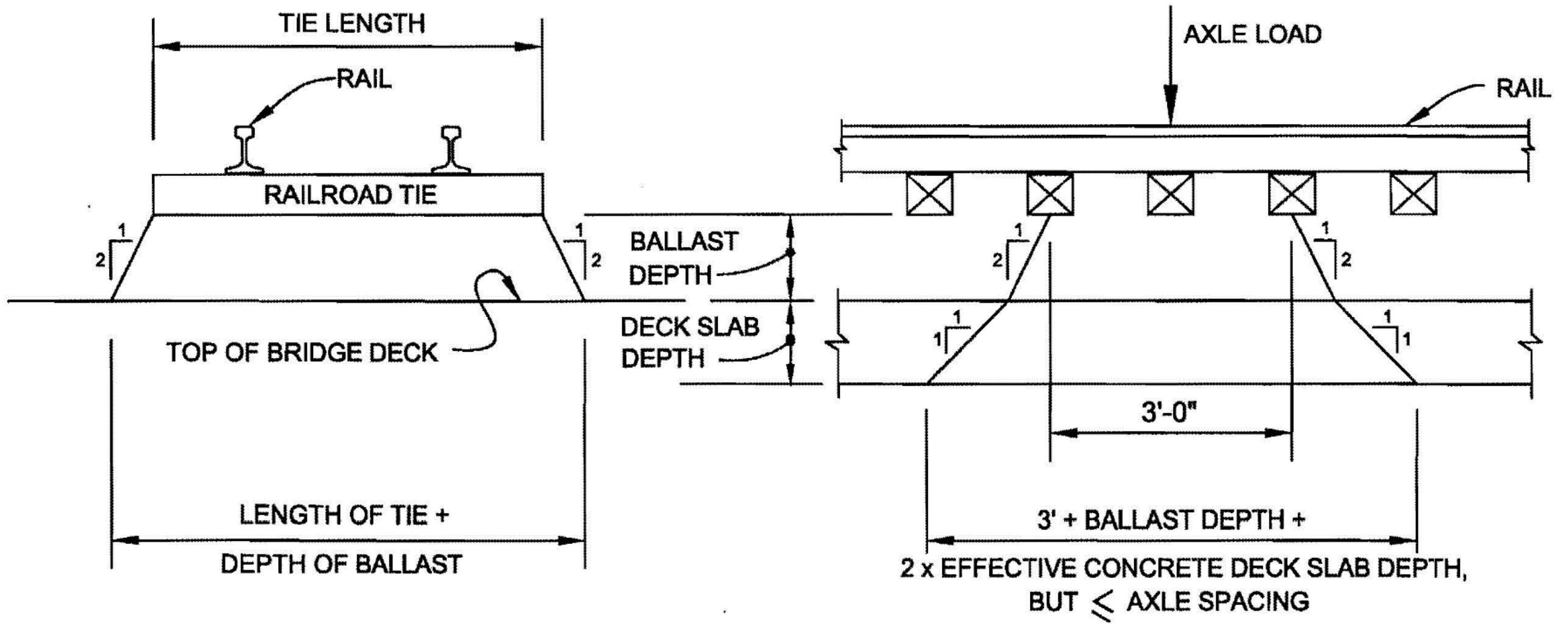
Design live load for all structures shall be Cooper E-80 loading as shown in the AREMA recommendations. The Cooper E-80 loading is a bridge loading and special care shall be exercised if this loading is to be used for other applications.

Refer to Figure 2.4 for information on the method to distribute the track load to top of a bridge deck for ballast deck structures.

Provide individual member load tables on the project drawings with the applied impact value and impact percentage shown. Design calculations shall clearly show how the loads and impact values were developed for the design.

All new bridges shall be designed for the seismic provisions for AREMA, Chapter 9 as modified by Chapter 4 of these guidelines.

Refer to Caltrain Standard Drawings for the typical PCJPB Track Roadbed Sections and to the previous transition details to develop the minimum and maximum ballast dead load for designing structures. The design shall use a minimum dead load of ballast of 13" (9" of ballast plus 4" of HMAC to be considered as all ballast for loading purposes) plus the depth of a tie. In addition a maximum ballast depth of 30" which includes the tie depth shall be used in design to account for future track raises due to track surfacing. Ballast dead load shall consider the superelevation present on curved track. The type of tie (wood, concrete, steel) used shall be considered in the determination of the dead load. Confirm with PCJPB the rail section and the type of ties (PCJPB prefers concrete ties) and fastening system to be used. Refer to Table 2.1 for suggested dead load values to be used for the calculations. The Commentary provides typical calculations for how the dead load weight of the track structure can be calculated.



FOR CONCRETE STRUCTURAL DESIGN :

LATERAL DISTRIBUTION OF LIVE LOAD TO BRIDGE DECK, PER AREMA CHAPTER 8.2.2.3.C (3)

LONGITUDINAL DISTRIBUTION OF LIVE LOAD TO BRIDGE DECK, PER AREMA CHAPTER 8.2.2.3.C (2)

FOR STEEL STRUCTURAL DESIGN DISTRIBUTION OF LIVE LOAD SEE AREMA CHAPTER 15

FOR TIMBER STRUCTURAL DESIGN DISTRIBUTION OF LIVE LOAD SEE AREMA CHAPTER 7

FIGURE 2.4 – TRACK LOAD DISTRIBUTION

TABLE 2.1 – DEAD LOADS

Material	Unit Weight
Track rails, inside guardrails and fastenings	200 lb/LF of track
Sand, gravel, or ballast (including timber track ties)	120 lb/ft^3
Reinforced concrete	150 lb/ft^3
Earthfilling materials	120 lb/ft^3
Waterproofing and protective covering	Estimated Weight
Timber	5 lb/foot board measure OR 60 lb/ft^3
Steel	490 lb/ft^3
Asphalt-mastic and bituminous macadam	150 lb/ft^3
Granite	170 lb/ft^3

Impact load for the structural material shall be per AREMA recommendations with the following clarification:

- Steel Bridges – The impact shall be considered for rolling equipment without hammer blow (freight and passenger cars, and locomotives other than steam).

- Reinforced Concrete – The impact load shall be considered for diesel engine impact as listed in Chapter 8, Part 2 of the AREMA Manual.

Impact loads shall be applied to the structure per AREMA recommendations and generally be applied as follows:

- **Group I** – Items in which Impact applies:
  1) The superstructure, including steel or concrete support columns, steel tower, legs of rigid frames, and generally those portions of the structure that extend to the main foundation.
  2) That portion of concrete or steel piles above the ground line where they are rigidly connected to the superstructure, as in rigid frames and continuous structures.
  3) That portion of cap or beam and supported piles in a spill through abutment or that are above the invert or roadway surface underneath the bridge.

- **Group II** – Items to which Impact does not apply:
  1) Abutments, retaining walls, wall type piers, and piles;
  2) Foundation piles and footings;
  3) Service walks.

Under normal working loads, some composite action may be expected between a concrete deck and its supporting steel members, whether or not special devices are furnished for shear transfer. For design purposes, however, use non-composite action to proportion the supporting steel members for E-65 live, impact, and dead loads and consider composite action for E-80 live, impact, and dead loads. The bottom of the deck slab shall be placed at least 1 inch below the top of supporting steel members. Composite action may be taken into account when satisfying the deflection-length ratio requirement of AREMA – Chapter 15, Part 1, Section 1.2.5 provided shear transfer devices are installed. For steel decks welded to the top flanges of multiple I-beam girders the composite action shall be considered only for deflection and not for the satisfying the strength of the girder.

On ballast deck structures the live load shall be distributed per the AREMA recommendations. See Figure 2.4 for typical load distribution. For precast, prestressed, single or double cell box girders this live load shall not be assumed to be distributed uniformly to all the boxes over the width of the bridge but shall be distributed to the individual box girders per the distribution in the AREMA recommendations. All precast box girders for a structure shall be designed for the same loading condition, taking into consideration that a track could be shifted anywhere on the deck in the future. An example would be for a single-track bridge where there where there are 8 single cell girders side by side in a bridge, but the normal distribution of the load per AREMA recommendation to the top of the girders would be distributed to only 4 of the girders. Therefore each of the 8 girders would be designed for the same loading as one of the girders in the 4 girder distribution of the live load. (In other words each single cell girder would not be designed for the track loading divided by the eight girders.) This requirement exists so that the track can be moved to any location on the bridge deck. It also assumes that no load sharing takes place between girders or that during the life of the structure any bond between that was designed into the girders may have been lost.

2.4 Clearances

2.4.1 General

The provisions for clearances along the PCJPB corridor are stated in this standard in the following sections. Consideration shall be given to the requirements of the following documents.

- PCJPB clearances as listed PCJPB Engineering Standards.
- AREMA – Manual for Railway Engineering, Chapter 28, Figure 28-1-2, Railway Bridges.
- Union Pacific Railroad – Standard Minimum Operation Clearances, Standard drawing 0038A. To be used on structures of the UPRR.
- In all cases the absolute minimum clearances stated in the California Public Utilities Commission General Orders (CPUC G.O.) shall be maintained on all structures.

Clearance diagrams shall be clearly shown on the design drawings.

2.4.2 Vertical Clearances

Provide sufficient vertical clearance and protection devices above the vehicle roadway surface to shield the railroad structure from collision of oversized and high loads. Projects shall be designed for minimum vertical clearance of 16’-6”. The vertical clearance shall be applied to the entire width of the roadway that is available for traffic. Due consideration shall be given to the need to apply overlays to the street surface and the above clearances adjusted accordingly.

All concrete structures that do not meet the minimum vertical clearance requirement shall have a collision impact device installed over the full width of the approaching travel lanes and attached to the bridge soffit (Clearances listed above shall include an allowance for the collision impact device); A suggested detail as shown in Figure 2.5 is to be applied on all new structures. Structures that require retrofit or rehabilitation shall be evaluated for the appropriate detail to use.

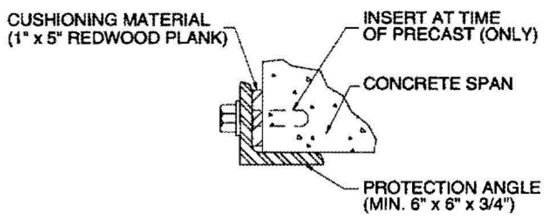


FIGURE 2.5 – COLLISION IMPACT DEVICE

All steel structures that do not meet the clearance requirements stated above shall have the bottom flange of the girder constructed of bolted elements (e.g., angles and plates).

All new railroad structures with clearance less than the clearances noted above shall have a sacrificial beam installed across the full width of the approaching travel lanes to the structure. Sacrificial beams shall be installed a minimum of 5 feet ahead of the structure and at the same elevation as the soffit of the railroad structure as shown in Figure 2.6. Sacrificial beam shall not carry any railroad loads and shall be anchored sufficiently to the bridge substructure. The sacrificial beam system shall be designed to withstand the impact of a vehicle collision and shall be prevented from falling onto the roadway surface. The sacrificial beam system can have aesthetic treatments as required as long as it does not interfere with the maintenance of the railroad structure. The retrofit of any structure with a sacrificial beam shall be on a case by case basis and in which other clearance options have been explored.

The PCJPB shall be formally notified prior to any resurfacing of the roadway or other work under a railroad structure. The notification shall include sufficient detail to determine if the clearance under the structure or the structure itself will be compromised or not. Final approval from the PCJPB shall be obtained prior to any work being performed.

The owner of the roadway shall be responsible for posting and maintaining structure sign vertical clearances and for any automatic advance street notification system as required.

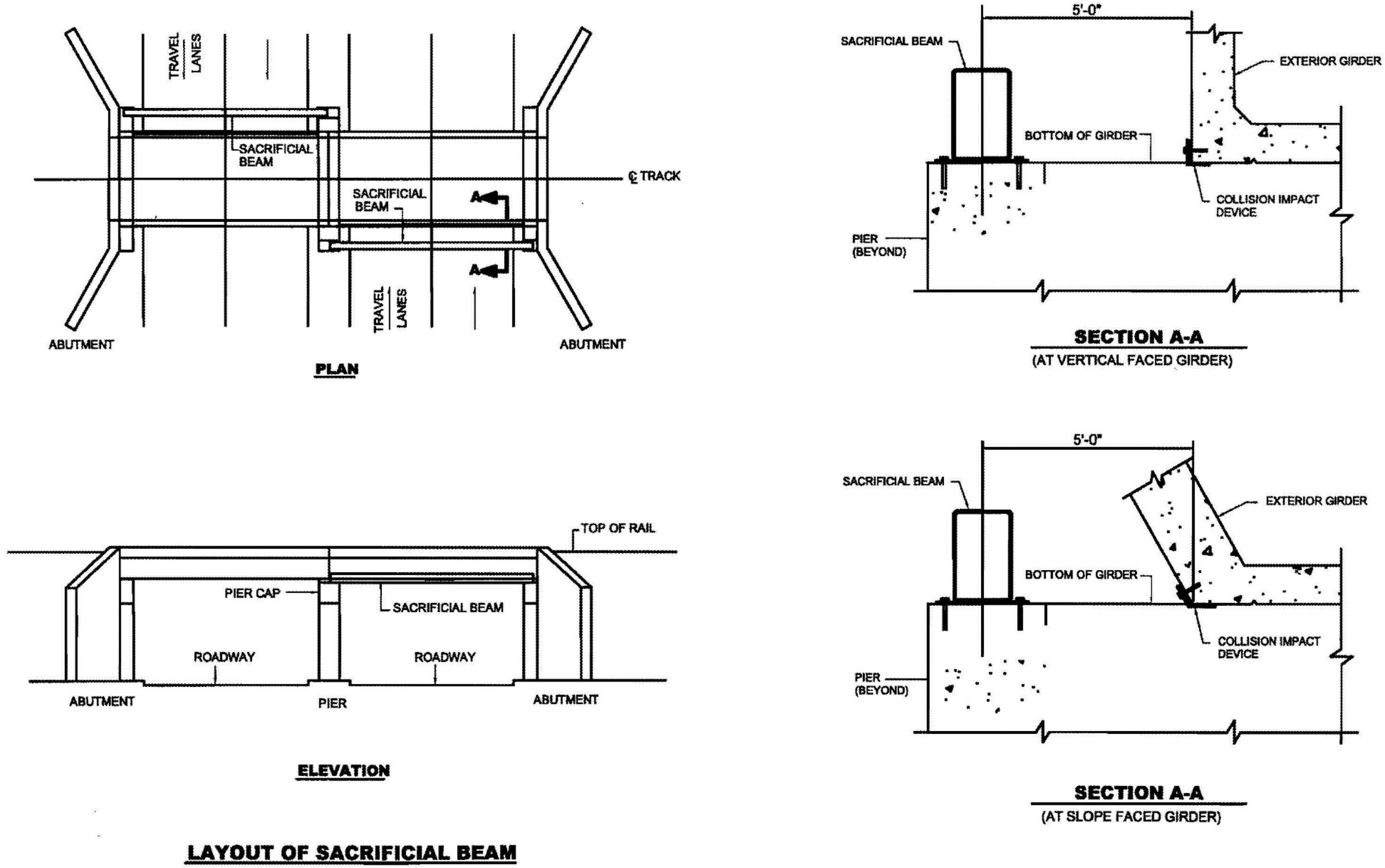


FIGURE 2.6 – SACRIFICIAL BEAM LAYOUT AND SECTION

The design for roadway profiles shall consider the future widening of the railroad bridge structure to accommodate additional tracks along the PCJPB corridor. Consult with the PCJPB to obtain clarification and direction on the PCJPB's future plans.

For structures over waterways the freeboard required shall be per the requirements of the agency that regulates the waterway, but in no case shall the freeboard be less than 2 feet clear above high water as defined by a 100-year flood event. Also, the top of the elevation of the track shall not be less than 4 feet above the 100-year flood event. If these parameters are not met then an enlarged structure opening or other solution shall be considered. For areas that involve the San Francisco Bay, such as for the Dumbarton line, the requirements for freeboard shall be handled on a case by case basis.

The preferred vertical clearance above any bridge structure is 25'-6" measured from top of rail. The minimum vertical shall be 24'-6" with an absolute minimum of 23'-6" only allowed under special circumstances.

2.4.3 Horizontal Clearances

The following minimum horizontal clearances on structures, measured from the centerline of track shall be as follows:

- To the face of railing, barrier, or fencing on bridges:
  - 10'-0" Minimum

- On structures at stations:
  - Per Caltrain Engineering Standards for platforms required for the loading and unloading passengers, etc.

2.5 Special Provisions

2.5.1 Concrete Structures

Substructure concrete shall have a minimum compressive strength of 4,000 psi at 28 days.

Superstructure concrete shall have a minimum compressive strength of 4,500 psi at 28 days.

Reinforcing Steel shall conform to ASTM A615 or A706 depending on the application. ASTM A706 reinforcing shall be used when detailing for seismic requirements.

When a cast-in-place concrete deck is to be used to retain the ballast on a steel or concrete structure the use of stay in place metal deck forms are not permitted. Stay in place forms do not allow inspection of the placed concrete deck soffit, can trap moisture leading to the corrosion of the steel form, and prevent maintenance and inspection of the bridge deck.

a) Precast Prestressed Box or AASHTO Girders

Manufactures of prestressed products shall be certified by the Prestress Concrete Institute Plant Certification Program in Product Group B4.

AREMA does not allow service load tension in prestressed concrete members.

Precast prestressed girders (Single or double cell boxes or AASHTO type) shall be designed with end and interior diaphragms. Interior diaphragms shall be spaced equally across the span length. Diaphragms spacing should not exceed 25 feet center to center. Provide the minimum number of diaphragms per span length as shown:

Span Length (in feet)	Number of Interior Diaphragms
35-50	1
51-75	2
Over 76	3

Where transverse tie rods are required they shall be installed at the end diaphragm and each interior diaphragm. Minimum size of tie rod shall be 1¼ inches in diameter. The tie rod assembly including all plates and hardware shall be hot dipped galvanized per ASTM A123 and A153.

Tie rod anchor assembly shall be recessed into the concrete and shall have a minimum of one-inch grout cover.

Strands at the ends of the precast prestressed members shall be cut one-inch minimum into the member and the resulting recessed pocket filled with grout.

Notching the ends of concrete girders or beams should not be considered.

For AASHTO beams the designer shall provide eighteen (18) inches clear between girder bottom flanges of beams to accommodate inspections and repairs.

Precast prestressed concrete box girders with interior voids shall be fabricated with witness holes in the bottom slab. Holes shall be located at the ends and middle of the girder and/or shall be located on either side of interior diaphragms.

b) Post-Tensioned Concrete Structures

Provisions are included here in case these types are finally selected for the bridge structure type and have approval from the PCJPB. Note that continuous cast-in-place structures are not preferred as previously mentioned in Section 2.2.

All post-tensioned concrete structure ducts shall be bonded (grouted).

i) Simple Span Structures

Precast Post-tensioned simple spans members, shall in any load condition, and during all stages of construction, require that the bottom fiber not develop any tension and the top fiber shall have at least a minimum compressive stress of 100 psi. At no time shall any variance occur either during construction or under any load configuration from these minimum requirements.

ii) Continuous Span Structures

Post-tensioned, continuous structures shall be designed for a minimum compressive force of 200 psi in the topmost regions of the element, and 50 psi minimum compressive force at lowermost regions of the element in the positive moment regions of the structure. In the negative moment regions of the structure the requirement will be reversed such that a minimum compressive force of 50 psi will be required in the topmost regions of the element and a minimum compressive force of 200 psi in the lower most regions of the element. These minimum compressive force requirements must be maintained during any stage of construction or any loading case.

2.5.2 Steel Structures

The minimum thickness for structural steel members (except for fillers) is ½ inch, including gusset plates for trusses.

High strength bolts shall be ASTM A325, type III with an allowable shear strength of 13.5 ksi for primary loads and 17.0 ksi for primary plus secondary loads. The minimum high strength bolt diameter is 7/8 inch. Prior approval from PCJPB is required for the use of A490 bolts, or the higher allowable values shown in AREMA.

The allowable bearing pressures on bearing surfaces for steel structures are given in the AREMA recommendations, Chapter 15.

All fracture critical members (FCM) and connections shall be designated on the project plans.

Floor beams shall be a minimum of 18 inches in depth, but must satisfy the stress and deflection requirements given in AREMA.

All exposed portions of the structure shall be designed to be accessible for inspection and maintenance. 18 inches is preferred clear spacing between flanges on parallel beams having depths greater than 36 inches.

Provisions for permanent jacking locations shall be provided on all steel spans to permit future maintenance of bridge bearings. Jacking locations shall be provided with jacking pads, jacking stiffeners, and connections capable of supporting the applicable dead load of the structure. A minimum of 12 inches shall be provided between the bridge seat and the bottom of any jacking surface to allow for placement of jacks.

All through plate girder and through truss type spans shall have an end floorbeam. Stringers at the end of a floor system, such as in a through truss or through plate girder type, shall be supported by an end floorbeam. Stringers shall not be allowed to be supported on independent bearings at the bridge ends.

All main load carrying steel members subject to tension stress shall meet the recommendation for notch toughness per AREMA.

AREMA recommendations do not use tension field design for steel girder webs.

Welded girders shall not be used where the vertical clearance is less than that stated in the vertical clearance section of these guidelines. If the vertical clearance is less than that allowed the bottom flange of girders shall consist of bolted construction (e.g., angles and cover plates).

All steel structures shall be painted except where galvanized. Painting shall conform to the current requirements of AASHTO or Caltrans specifications and recommendations of the Steel Structures

Painting Council Manual (SSPC). Paint shall be applied in accordance with the Manufacturer's recommendation or in compliance with the recommendation of SSPC, whichever is most restrictive.

For all steel structures, the provisions for quality of workmanship in fabrication require that the fabricator shall be currently an AISC Certified Steel Bridge Fabricator - Advanced (ABR). Additionally, the fabricator shall currently have the following endorsements if they apply to the type of structure being fabricated, Fracture Critical Endorsement (F) and/or Sophisticated Paint Endorsement (P).

2.5.3 Ballast Deck Bridge Structures

For a tangent single-track bridge structure the width of the deck shall not be less than 20 feet for bridges without separate walkways or grating walkways, measured from inside face of curb to inside face of curb. The clear distance from centerline of track to inside face of curb shall not be less than 10 feet. The minimum ballast trough width from centerline of track may be 7 feet 6 inches if a solid walkway (e.g., concrete) is used which will also include a curb and toe-board incorporated into the walkway.

The top of ballast curb shall be approximately the same elevation as the base of the highest rail plus a minimum of 9 inches. The additional height will accommodate future track raises and must be high enough to keep ballast from spilling over and off the bridge when these track raises are made. In addition other suitable means can be provided to prevent ballast from falling off bridge decks, such as providing additional screening 1'-0" high along the bridge edges.

Walkways shall be provided on both sides of the deck. Walkways shall not be less than 2 feet wide. Where structural members (such as floor beam knee braces) encroach into the walkway area it shall not be considered as an obstruction to the walkway (see CPUC GO 26-D). Ballast walkways are preferred, especially on bridges less than 100 feet long, to allow for flexibility in future track relocations and to allow placement of signal cables in the ballast section. If necessary and with the PCJPB's concurrence the walkways can be either concrete or metal grating. Ballast decks shall be designed to allow tracks to be moved anywhere on the deck. Ballast retainers and curbs shall not be constructed between tracks, unless they are for temporary use.

Guardrails shall be provided on both sides of the deck. Guardrails shall be designed to require a minimum of maintenance and not impinge on the minimum clearance requirements. PCJPB recommends the following types of guardrails, which will depend on the location where it is to be installed:

1) Chain link railing (6 feet high) with a 1-inch by 1-inch opening (1-inch mesh). Suitable for pedestrian overhead and over freeways.
2) Tubular railing. Suitable over local streets and highways.
3) Picket railing. Suitable over local streets and highways.
4) 3/8-inch diameter cable railing with a minimum of 3 separate cables. Suitable over water courses and local streets.

All ballast troughs shall be sloped transversely not less than 0.5% for drainage. Low points in the ballast trough shall not occur under a track and shall be located not less than 7 feet from the centerline of track. A longitudinal and transverse collection system shall be provided to transfer drainage water off of the bridge structure away from the track roadbed to an approved discharge location.

All composite decks, cast-in-place concrete decks, and steel ballast pans shall be waterproofed when the HMAC underlayment is not provided on the deck. When the HMAC underlayment is used the bridge joints and expansion joints shall be waterproofed (e.g., a possible waterproofing system is the MEL-DEK system manufactured by W. R. Meadows). Structure ballast decks located over roadways or pedestrian walkways shall be waterproofed. Approval from the PCJPB shall be obtained to eliminate the HMAC on bridge decks. All top slabs of precast prestressed concrete structures and concrete decks that do not require waterproofing shall be sealed with an approved concrete sealer. Waterproofing systems shall conform to AREMA recommendations (Chapter 29) and shall be approved by PCJPB. Butyl Rubber or EPDM membranes with asphaltic protection panels (minimum of 2 layers for a total thickness of 1 inch) or spray on membranes (e.g. “Eliminator” as manufactured by Stirling-Lloyd) are waterproofing systems that have been used in the past. All bridge deck joints shall be detailed to prevent the intrusion of ballast, HMAC, or other foreign materials into the joint (e.g., steel plates over joint) and still provide for bridge movement at expansion joints.

For ballast deck bridges the transition from the bridge to the track roadbed shall be designed to prevent ballast from spilling over. Ballast shall be prevented from spilling onto the tops of abutments and piers and kept away from bridge bearings. Ballast shall be prevented from spilling onto highways and walkways below bridge structures. Ballast shall be prevented from spilling down the track roadbed embankment behind bridge abutments. Appropriate wingwalls and breast boards shall be provided to contain the ballast at the transition from bridge to track roadbed. Walkway requirements shall be extending off the bridge structure.

2.5.4 Railroad Electrification

PCJPB/Caltrain is currently electrifying the corridor. All structures shall provide for a future overhead catenary system. Space for catenary support poles along the right-of-way and on structures shall be provided for. Overhead (vertical) clearance and horizontal clearance shall comply with the following references: CPUC GO 26-D and GO95, NESC, NEC, AREMA Chapters 28 and 33, and CPUC Resolution SED-2 dated November 10, 2016.

All concrete superstructures and substructures shall be detailed to mitigate the effects of stray current corrosion of steel reinforcing, prestressing elements, and other steel components. This will require that electrical continuity be provided between all steel elements within each concrete structural component and be run to a central location at time the structure is designed. When the corridor is electrified the central location points will be connected to the corrosion control system. Comply with GO95, AREMA Chapter 33, IEEE, NESC, and NEC provisions for stray current.

2.6 Substructure

The substructure elements shall be designed in accordance with appropriate sections of AREMA and per Caltrans. In addition to this section refer to Chapter 6 Retaining Walls of these standards for further information on abutments.

2.6.1 General Layout

The abutments and wingwalls shall be wide enough to support the standard PCJPB track roadbed section from shoulder to shoulder. Abutments/Wingwalls shall provide for the support of the sacrificial beam when required. The abutment width shall be sufficient to provide for a 13-foot shoulder on both sides measured from the centerline of the track. In multiple track areas the minimum track centers shall be considered as 15 feet for use in determining abutment widths. Track centers and future tracks shall be discussed with PCJPB prior to design. Wingwalls shall be designed to support the PCJPB standard embankment slope shown in the Caltrain Engineering Standards.

Handrails for bridges shall be returned on the backwall of the abutment and/or wingwalls.

Provide a minimum of 12 inches from the edge of the masonry plate or shoe to the edge of abutment or pier seat. Also, a minimum of 12 inches shall be provided vertically between the bridge seat and bottom of all beams and girders to allow for future jacking. If this jacking area is not possible, jacking pockets with a 12-inch by 12-inch seat shall be provided in front of the bridge bearing.

Slope top of abutment and pier to drain. If weathering steel is used in the superstructure the top of the abutment or pier seat shall incorporate a method for collecting and disposing of drainage water in order not to stain the exposed face of the abutment or pier.

Piers with two columns or a solid pier wall are preferred over single column piers.

2.6.2 Geotechnical Investigation

Each bridge location shall have an investigation into the soils at a bridge site. Refer to AREMA Manual Chapter 22 for guidance on geotechnical information required for railroad bridges.

Depending on the nature and importance of the bridge work required the designer shall at the least research the existing geotechnical information available. Information may be obtained from the PCJPB or from other public agencies. This is all that may be required for simple structures. For more complex structures and new bridges the investigation may require boring to characterize the nature of the soils and foundation type recommendations at each abutment and pier. It is recommended that at least one boring occur at each abutment and pier location for new structures.

The geotech report shall include an evaluation of the geological formations and soils at the project site. It shall also describe and evaluate any seismic hazards that are present, such as expected ground shaking that can occur and the potential for liquefaction. The report shall give recommendations on the type of foundation suitable for the site and the structure and provide seismic data, such as peak ground acceleration, that should be used. Recommendations shall, include the recommended elevations for spread footings, pile type and tip elevations.

The geotech report shall contain any special requirements regarding potential constructability problems including caving, soil compaction issues, variations that may be expected in pile driving, and any groundwater issues, etc. The geotech shall recommend any job specific specification that should be followed as part of the report.

2.6.3 Foundation Types

Structure foundations can generally be classified in the following categories: (1) footing foundations (also referred to as spread footing), (2) pile supported foundations (driven or non-driven piles); (3) special foundation types which would include pier columns and tiedowns.

For pile supported foundations, it is the designer's responsibility to select the pile type consistent with the Foundation Report's recommendations. Also, the selected pile type should fulfill the requirements for economy, competitive bidding, and availability for the particular conditions prevailing at the site, especially the PCJPB train operating conditions that will occur during foundation installation.

In cases where both footing and pile foundations are applicable, existing field conditions as well as economics will determine the foundation type. While the foundation type selection is primarily determined by the geological nature of the foundation material itself, non-geologic features are considered in the selection and design of structure foundations.

A seal course is frequently specified as a foundation aid when water problems are anticipated. Seal course concrete can be placed under water, the general purpose being to seal the bottom of a tight cofferdam against hydrostatic pressure. This enables dewatering of the cofferdam and construction of the footing "in the dry."

Generally, footing foundations are more economical than pile supported foundations. CIDH concrete piles are the most economical pile supported foundation with steel piles being the most expensive.

Pile type comparisons may be found in Table 2.2.

Spread Footing Foundations

Footing foundations transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing; whereas pile supported foundations transmit design loads through pile friction, end bearing or both.

Since the load bearing capacity of most soils is relatively low, footing areas will be large in relation to the cross section of the supported member, i.e., columns.

Each individual footing foundation must be sized so that the maximum soil bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. In addition, footing settlement must not exceed tolerable limits established for differential and total settlement as established by the geotech. Refer to Design Manual 7.02 – Foundations and Earth Structures, NAVFAC DM-7.02 for guidance on the ·settlement of structures. Each footing foundation must also be structurally capable of spreading design loads laterally over the entire footing area.

Footing foundations can be classified into two general categories: (1) footings that support a single structural member, spread footings, and (2) footings that support two or more structural members, combined footings.

Also refer to Chapter 6 on retaining walls.

TABLE 2.2 – PILE TYPES

Pile Types	Descriptions	Geologic Factors	Nongeologic Factors
Footing Foundations	Virtually unlimited in use	The soil profile, the location of the water table and any potential fluctuation, and the potential for scour or undermining	The size and shape of the footing, adjacent structures, and existing utilities
Drive Piles	Used where foundation material will not support a footing foundation or discourages the use of a Cast-In-Drilled Hole (CIDH) concrete pile. Pile types are precast concrete, steel structural sections, steel pipe and timber.	Soil profile, driving difficulties and corrosive soil problems	Adjacent structures, existing utilities, required pile length, restricted overhead clearances, accessibility and noise restrictions
Non-driven Piles	Consists of CIDH concrete piles and alternative footing design piles, i.e., GeoJet Tubex, Pin Piles. CIDH piles are extensively used where piles are required and foundation conditions permit their use. The slurry displacement method of construction of CIDH piles is used where driven piles are impractical and ground conditions necessitate its use. Alternative footing design piles are used when conditions warrant their use.	Location of water table and potential fluctuation, and the soil profile	Adjacent structures, existing utilities, restricted overhead clearances and accessibility
Special Case: Tiedowns or Tension Piles	Used typically for seismic retrofitting of existing footing where overturning needs to be addressed, or specially needs.	Location of water table and potential fluctuation, and the soil profile	Adjacent structures, existing utilities, restricted overhead clearances and accessibility

Driven Concrete Piles

Driven concrete piles come in a variety of sizes, shapes and methods of construction. In cross section, they can be square, octagonal, round, solid or hollow. These piles generally vary in sizes from 10 to 60 inches. They can be either conventionally reinforced or prestressed (most common). They can also be either precast (most common) or they can be cast in driven steel shells. The types of steel shells vary from 10 to 18 inches in diameter for heavy walled pile which are driven directly with the hammer, to thin walled or step-taper pipes which are driven with a mandrel. The steel shell may have a flat bottom or be pointed, and may be step-tapered or a uniform section (such as a Monotube pile). Splicing precast concrete piles is difficult, time consuming, and expensive. Hence, when excessively long piles are required to obtain necessary bearing, precast piles may be precluded.

The unit cost to furnish concrete piles is usually lower than the steel equivalent. But this cost is often offset by the requirement for a larger crane and hammer to handle the heavier pile. This is particularly true when there are a small number of piles to drive.

Cast-in-Steel-Shell (CISS) piles are typically large diameter reinforced concrete piles cast in driven steel shells. Because of the large diameter of the driven element, steel shells are typically driven open-ended. The project geotechnical engineer needs to recommend the required soil plug to adequately resist the hydrostatic pressure.

Driven Steel Piles

Steel piling includes “H” piles and pipe piles (empty or concrete filled). The pipe section is a standard alternate for precast piles, but is seldom selected. Steel pipe piles for the PCJPB shall be concrete filled and have reinforcing bar cages installed in the pile that run into the pile cap for attachment.

Although steel piling is relative expensive on a per foot furnish basis, it has a number of advantages. They come in sizes varying from HP 8x36 to HP 14x117 rolled shapes or may consist of structural steel plates welded together. They are available in high strength and corrosion resistant steels. They can penetrate to bedrock where other piles would be destroyed by driving. However, even with “H” piles, care must be taken when long duration hard driving is encountered as the pile tips can be damaged or the intended penetration path of the pile can be drastically deflected. Some of this type of damage can be prevented by using a reinforced point on the pile. Due to the light weight and ease of splicing, they are useful where great depths of unstable material must be penetrated before reaching the desire load carrying stratum and in locations where reduced clearances required use of short sections. They are useful where piles must be closely spaced to carry a heavy load because they displace a minimal amount of material when driven.

Splice details should be shown in contract plans. Pile welding work requires special attention and various methods can be used to pre-qualify welders who will be performing the work.

Sometimes “H” piles (and can apply to all pile types) must be driven below the specified tip elevation before minimum bearing is attained. This can present a construction administrative problem (cost) if the length driven below the specified tip elevation is significant. Steel lugs welded to the piles are commonly used to solve this problem.

Driven Timber Piles

Although rarely used in modern permanent construction, treated timber piles are commonly found in existing railroad trestles, foundations below masonry and concrete piers, and in protective fenders around piers in navigable waterways. Untreated timber piles are still occasionally used for temporary construction, such as shoring and falsework.

Even treated timber is vulnerable to decay, except underground where oxygen is in short supply. Unless protected with some sort of jacket or protective wrap, marine borers in salt or brackish water can damage the piles. On the other hand, timber is inherently resistant to impact loading and fatigue. Treated timber piles may be used for fender systems or temporary construction only.

The maximum allowable bearing capacity of a timber piles is typically limited to around 45 tons, whereas most steel or concrete piles will support at least 75 tons. Timber piles are difficult to splice, and thus are usually used as single unspliced piles under 60 feet in length. They are not suitable where a deeper penetration is required.

Railroads typical have many existing timber pile structures in use and are phasing out timber trestles as funds become available, typically replacing them with precast concrete girder bridges on steel or concrete piles. Many timber trestles are still maintained in service, however. Furthermore, concrete and masonry piers founded on timber piles are very common and continue to perform well because buried piles, practically sealed from an appreciable supply of oxygen, do not decay.

Alternative Piles

These piles may be used for special applications. They are: the GeoJet Foundation Unit, the Tubex Grout Injection Unit, the Nicholson Pin Pile, and the Soil-Cement Pile. The GeoJet Foundation Unit consists of a structural member inserted into an augured soil cement column. The Tubex Grout Injection Unit is a steel pile with a special cast iron tip filled with concrete surrounded by an injected grout/soil mixture layer. The Nicholson Pin Pile is a bored cast-in-place pile with a reinforcing bar in a grouted hole. Soil-Cement piles have successfully been used in several projects along the PCJPB corridor. Originally used for shoring tunnel walls during cut-and-cover construction for BART and VTA, these piles have performed well in railroad bridge abutments and walls of pedestrian tunnels under PCJPB tracks.

Construction of soil-cement piles on PCJPB projects has been to first run an auger, typically 24 to 42 inches in diameter, into the ground. Cement grout is injected into the soil from nozzles on the auger as it is backed out of the ground, creating a column of mixed soil and grout. An HP pile or wide flange section is vibrated into the mixture while it is still soft. The pile unit becomes functional as part of a retaining wall or as a load bearing pile after the soil-cement mixture gains strength. Depending on the end product a cap can be welded or cast onto the top of several piles.

Cast-in-Drilled Hole Piles

These piles are simply reinforced concrete piles cast in holes drilled to predetermined elevations. Diameters range from 12 to 126 inches and lengths range from 10 feet to over 120 feet. They are satisfactory in suitable material and are generally more economical than most other types of piling. They are especially advantageous where vibration from a pile driver might damage adjacent structures such as pipelines, etc.

Much experience has been gained with this pile type because of their extensive use in the construction of bridge structures. While they are the most economical of all commonly used piles, their use is generally limited to certain ground conditions.

The ground formation in which the holes for CIDH piles are to be drilled must be of such a nature that the drilled holes will retain their shape and will not cave in when concrete is placed. Because of cave-in and concrete placement difficulties, these piles are not recommended for use as battered piles. Nor are they recommended where groundwater is present, unless dewatering can be done without unreasonable effort and unless concrete can be placed with a casing that is removed during construction or a casing that remains in place (a type of CISS pile). If groundwater or caving conditions are present, the piles can be constructed by the slurry displacement method if permitted in the contract specifications.

2.7 Construction Specifications

Technical Specifications for bridge construction shall comply with the following:

- PCJPB recommendations presented in these guidelines
- AREMA – Manual of Railway Engineering, Volume 2, Structures
- Caltrans Standard Specifications
- Standard Specification for Public Works Construction
- American Association for State Highway and Transportation Officials Specifications

2.8 Miscellaneous

PCJPB projects shall show all primary dimensions in drawings for bridges and underpass grade separation structures in English units. Projects that require the use of metric units shall indicate all dimensions and design criteria assumptions in dual units on the project general plan sheet. English units are to be shown in parentheses. Primary dimensions refer to but are not limited to the following:

a) Horizontal and vertical clearances.

b) Track spacing, track stationing, and railroad right-of-way.

c) Top of rail elevation over structure and grade profile.

d) Span length, width and depth of superstructure elements.

e) Size and limits for barriers and fences.

f) Location and elevation of underground or aerial utilities and their relocation adjustments if required.

g) Size, elevation and location of pier or abutment footings for spans adjacent to railroad tracks.

h) Size of structure supports (pier or abutment walls, columns}.

i) Size and elevations of pier protection walls, if required.

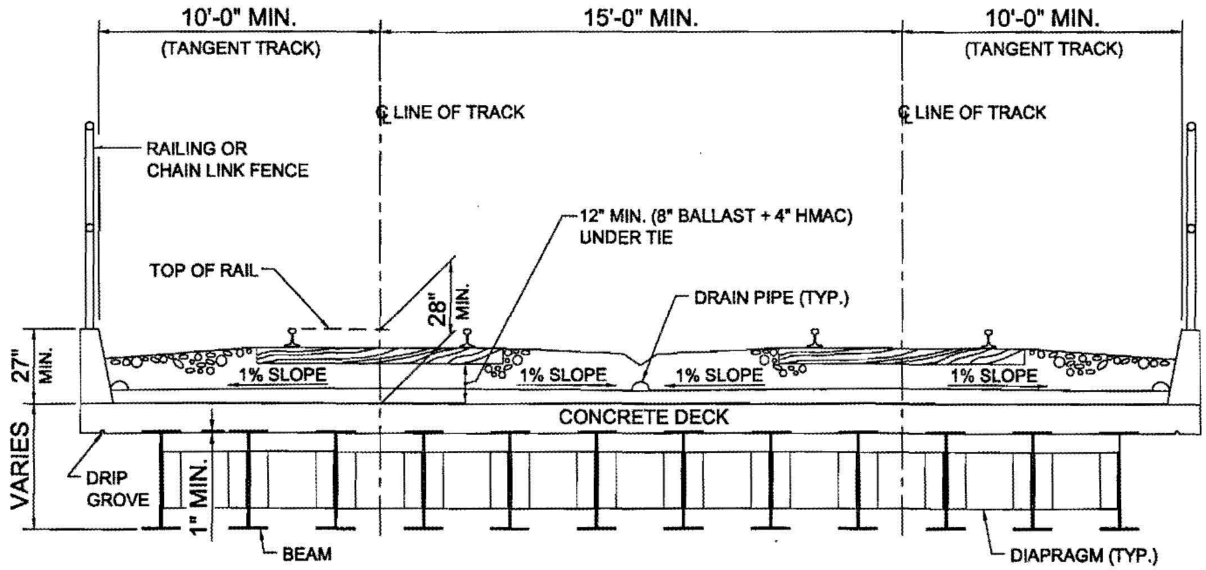
j) Shoring location and limits, if required.

k) Size and location of drainage structures and ditches.

l) Temporary construction vertical or horizontal clearances, if required.

All estimates for bridge construction in addition to the total cost shall have the bridge cost expressed in track feet. Track foot cost shall be computed by the following formula:

$$\frac{\textit{Total Railroad Bridge Cost}}{\textit{Bridge Length by Number of Tracks}}$$



STEEL BEAMS (MIN 4 PER TRACK)

NO SCALE

NOTES:

1. SHOWN WITH CIP CONCRETE DECK. A STEEL DECK COULD ALSO BE USED.
2. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
3. COST RANK 7/10.

SPAN RANGE

0'– 50' STEEL ROLLED BEAMS
50'–150' STEEL PLATE GIRDERS (MIN. 4 BEAMS PER TRACK)

DEPTH (SOFFIT TO T/R)

0.08 x SPAN + 2.90' (CONCRETE DECK)
0.08 x SPAN + 2.45' (STEEL DECK)

ADVANTAGES

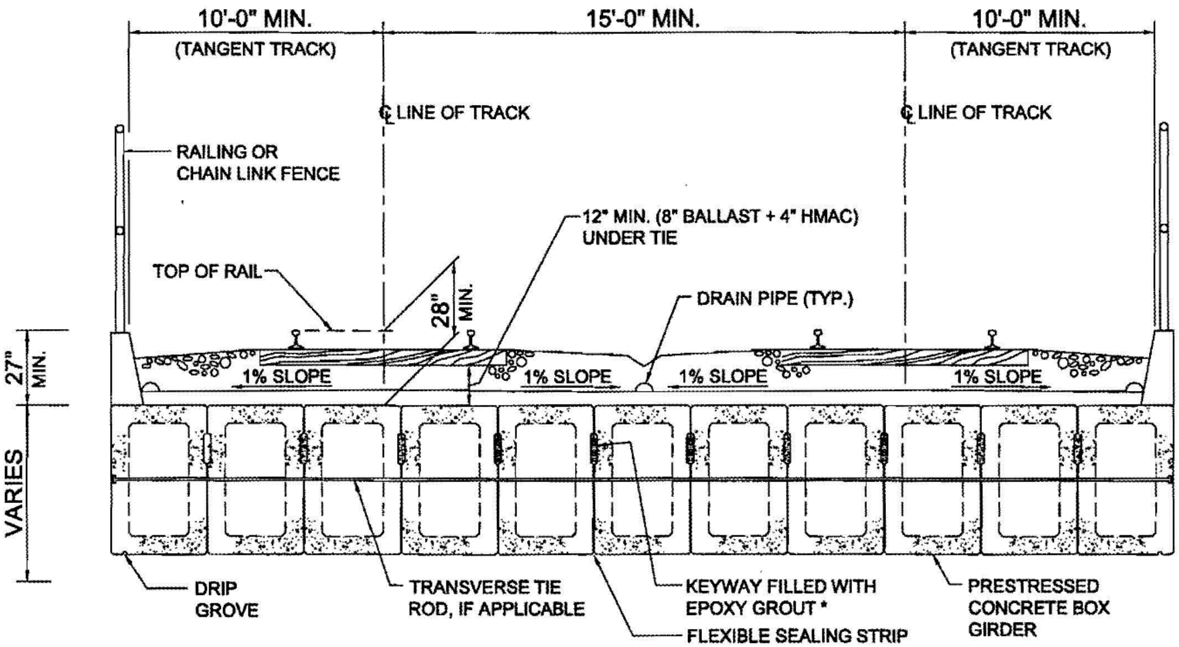
- SIMPLE CONSTRUCTION

DISADVANTAGES

- REQUIRES SEPARATE CONCRETE DECK PLACEMENT

THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

FIGURE 2.7 – STEEL BEAM SPAN



NOTES:

1. SHOWN WITH OPTIONAL CONCRETE DECK.
2. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
3. COST RANK 5–6/10.

* IF APPLICABLE, MAY NOT BE REQUIRED ON BRIDGES OVER CREEK AND WITH DOUBLE CELL PRESTRESSED/PRECAST BOX GIRDERS.

PRESTRESSED/PRECAST GIRDER SPANS OVER ROADWAYS, PEDESTRIAN UNDERPASS SHALL HAVE A WATERPROOF DECK.

PRECAST CONCRETE BOX GIRDERS

NO SCALE

SPAN RANGE

0'–20'	SOLID SLAB GIRDER (NO VOID)
20'–44'	DOUBLE BOX GIRDER
44'–70'	SINGLE BOX GIRDER

DEPTH (SOFFIT TO T/R)

0.08 x SPAN + 2.30' (WITHOUT CONCRETE DECK)

THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

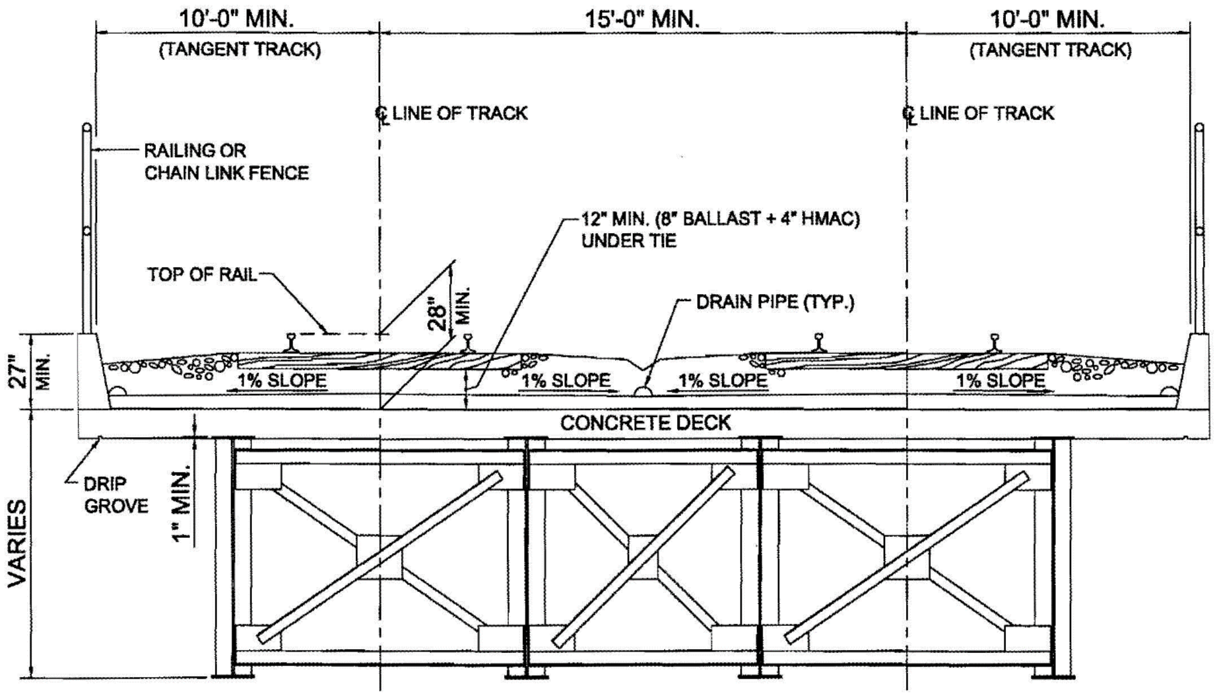
ADVANTAGES

- SIMPLE CONSTRUCTION
- EASY TO REPLACE
- NO SEPARATE DECK POUR REQUIRED
- FALSEWORK NOT REQUIRED DURING CONSTRUCTION.

DISADVANTAGES

- HIGHER COST FOR A CONCRETE BRIDGE

FIGURE 2.8 – PRESTRESSED PRECAST CONCRETE BOX GIRDER SPAN



NOTES:

1. SHOWN WITH CIP CONCRETE DECK. A STEEL DECK COULD ALSO BE USED.
2. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
3. COST RANK 8/10.

STEEL DECK PLATE GIRDERS (2 PER TRACK)

NO SCALE

SPAN RANGE

50'–150' (2 GIRDERS PER TRACK)

DEPTH (SOFFIT TO T/R)

0.10 x SPAN + 2.90' (CONCRETE DECK)
0.10 x SPAN + 2.45' (STEEL DECK)

ADVANTAGES

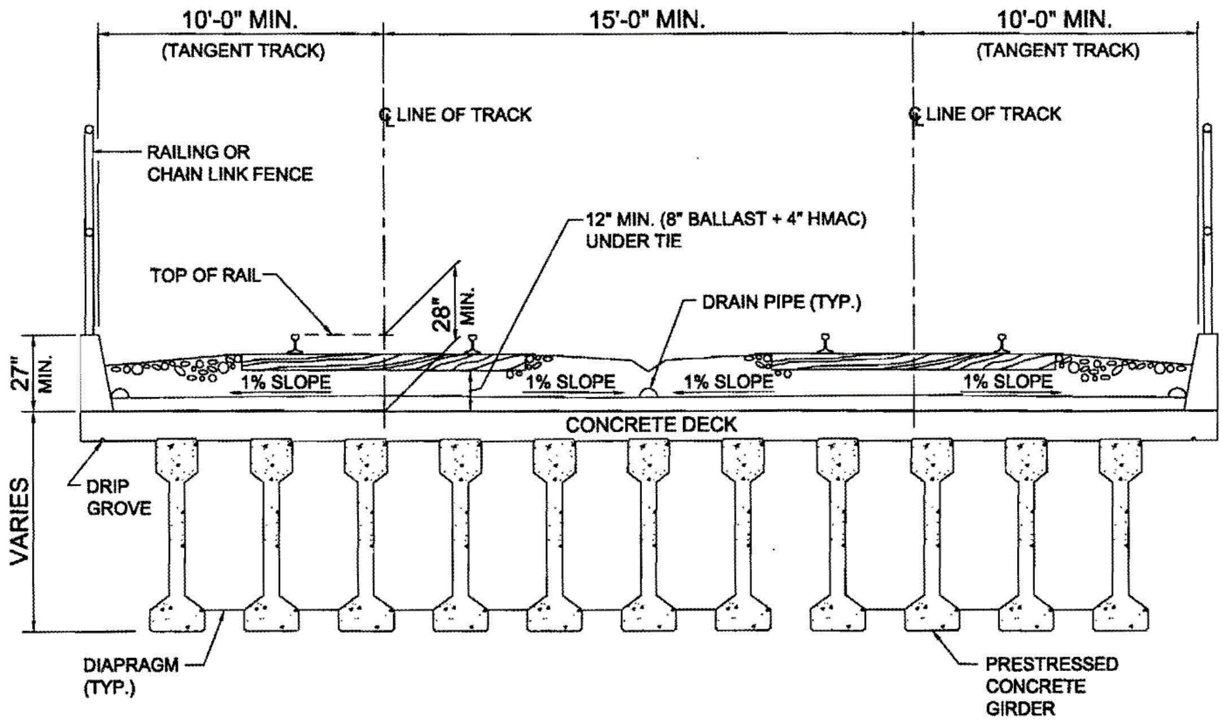
- SIMPLE CONSTRUCTION

DISADVANTAGES

- REQUIRES SEPARATE CONCRETE DECK PLACEMENT
- SUPERSTRUCTURE NOT AS REDUNDANT
- FRACTURE CRITICAL DESIGN – CONSIDER BOLTED BOTTOM FLANGE TO WEB DETAIL.

THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

FIGURE 2.9 – STEEL DECK PLATE GIRDER SPAN



PRECAST CONCRETE BEAMS WITH CAST IN PLACE CONCRETE DECK

NO SCALE

NOTES:

1. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
2. COST RANK 4/10.

SPAN RANGE

20'–70'

DEPTH (SOFFIT TO T/R)

0.08 x SPAN + 2.90'

THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

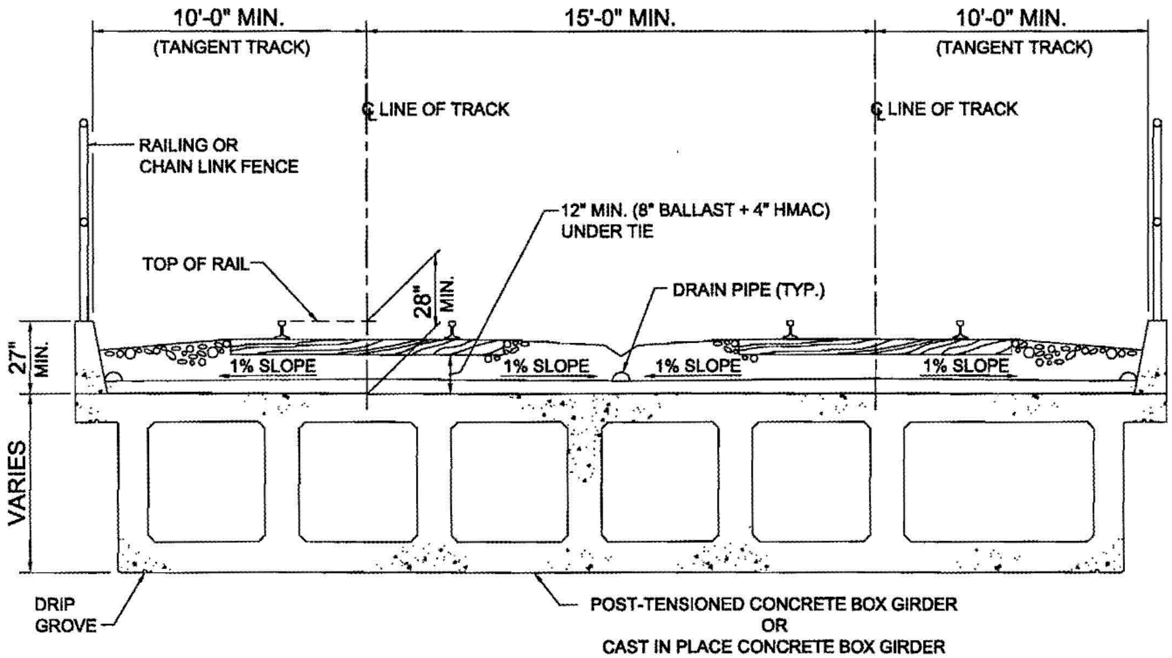
ADVANTAGES

- LOW COST
- EASY TO REPLACE

DISADVANTAGES

- REQUIRES SEPARATE CONCRETE DECK AND DIAPHRAM PLACEMENT

FIGURE 2.10 – PRESTRESSED PRECAST CONCRETE AASHTO TYPE BEAM SPAN WITH CONCRETE DECK



NOTES:

1. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
2. COST RANK 2/10.

CONVENTIONAL REINFORCED CONCRETE BOX GIRDERS

CAST-IN-PLACE POST-TENSIONED CONCRETE BOX GIRDERS

NO SCALE

SPAN RANGE	DEPTH (SOFFIT TO T/R)
50'–250'	0.08 x SPAN + 2.30'

ADVANTAGES

- LOW COST
- FAMILIAR CONSTRUCTION PROCEDURES IN CALIFORNIA

DISADVANTAGES

- LONG CONSTRUCTION TIME
- REQUIRES FALSEWORK
- PRECAST PREFERRED BY RAILROADS

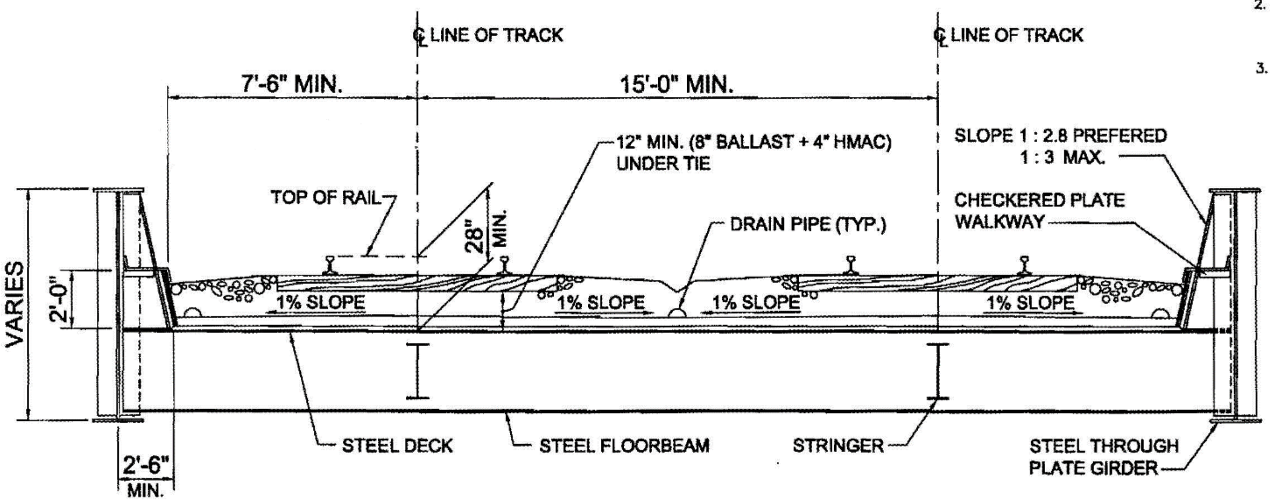
THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

CAST-IN-PLACE CONCRETE BOX GIRDER SPAN CONVENTIONAL REINFORCED

FIGURE 2.11 – CAST-IN-PLACE POST-TENSIONED CONCRETE BOX GIRDER SPAN

NOTES:

1. STEEL DECK SHOWN, CONCRETE DECK COULD ALSO USED.
2. DOUBLE TRACK SHOWN, OTHER TRACK ARRANGEMENTS MAY BE REQUIRED.
3. COST RANK 9/10 THROUGH GIRDER
   10/10 THROUGH TRUSS



STEEL THROUGH PLATE GIRDERS

NO SCALE

SPAN RANGE

50'–150' THROUGH GIRDER (TG)
150'–500' THROUGH TRUSS

DEPTH (SOFFIT TO T/R)

0.08 x SPAN (MAIN GIRDERS, SINGLE TRACK–TG)
0.095 x FLOORBEAM LENGTH (MIN 18") + 2.5' (STEEL DECK–TG)
0.095 x FLOORBEAM LENGTH (MIN 18") + 3.0' (CONC DECK–TG)
7'± FOR THROUGH TRUSS

THE MINIMUM DEPTHS SHOWN ARE RECOMMENDED FOR PLANNING STAGE DEVELOPMENT. LESSER DEPTHS MAY BE USED IF AREMA REQUIREMENTS FOR STRESS AND DEFLECTION ARE SATISFIED.

ADVANTAGES

- LOW PROFILE FOR SPAN LENGTH (SLIGHTLY HIGHER COST W/CONCRETE DECK)

DISADVANTAGES

- SUSCEPTIBLE TO DAMAGE
- REPLACEMENT SUBJECT TO LONG LEAD TIME
- LIMITED TO ONE TRACK BETWEEN GIRDERS (TWO POSSIBLE WITH DEPTH INCREASE)

FIGURE 2.12 – STEEL THROUGH PLATE GIRDER SPAN

CHAPTER 3

GRADE SEPARATIONS OVER RAILROAD

CHAPTER 3: GRADE SEPARATIONS OVER RAILROAD

Purpose

This guideline was created to provide standards and requirements regarding design and construction of new or modified existing grade separation overhead structures that affect the tracks and property of the Peninsula Corridor Joint Powers Board (PCJPB). Design engineers should use these guidelines as the basis for preliminary design and final design.

3.1 General Requirements

Design and construction of overhead grade separation structures shall comply with Figure 3.1.

Changes in railroad horizontal alignment and profile should be avoided unless they can be shown to benefit the railroad. The PCJPB Deputy Director of Engineering must approve any alignment or profile changes.

Provisions should be made for four tracks along the PCJPB Right-of-Way where feasible.

If the overhead structure occurs within curve limits, the minimum track centers, as shown on the figures in this document shall be increased by 1.5 inches per degree of curve.

Right of Entry, Right-of-Way encroachment and Construction and Maintenance (C&M) agreements consistent with PCJPB policy will be required.

The municipality or other public agency must make application to the California Public Utilities Commission (CPUC) for the grade separation. The PCJPB staff will support the application, if required, provided that it is consistent with the PCJPB's future plans.

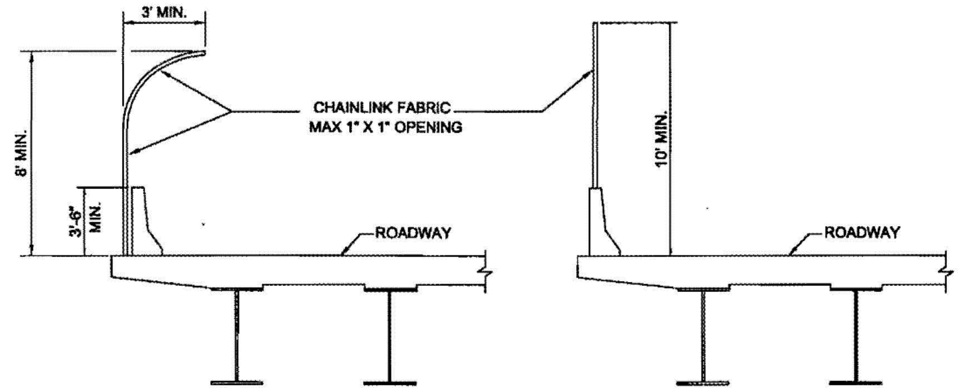
For submittal requirements, see Chapter 7.

The agency or their representative shall provide the basic information requested on the Overhead Grade Separation Data Sheet to PCJPB at the initial stage of the project. A copy of the Overhead Grade Separation Data Sheet is included in Chapter 7.

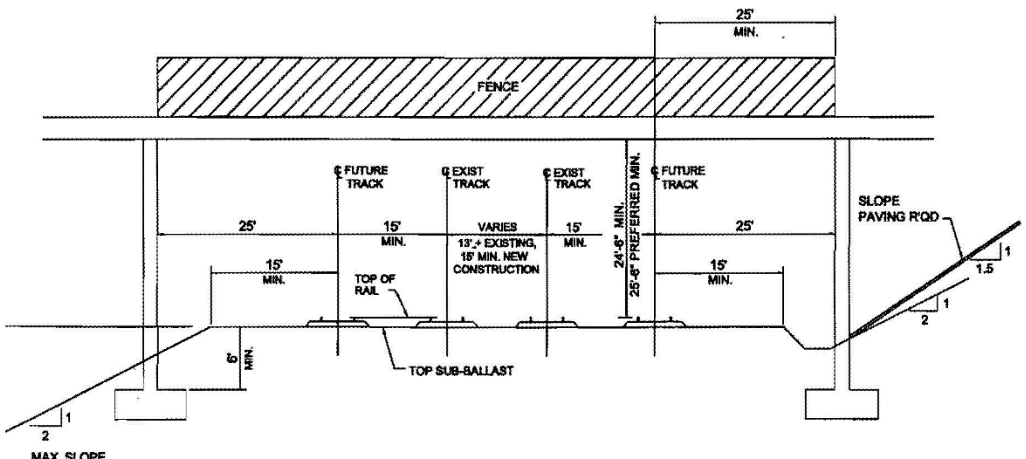
3.2 Superstructure

Erection of the superstructure over the tracks shall not interfere with railroad operations unless approved by the Deputy Director of Engineering and the Director of Rail Operations. Precast girders or prefabricated spans are preferred for spans over tracks to minimize track outages.

Fences or other methods shall be provided on both sides of all overhead structures to protect railroad facilities and the safety of railroad employees below from objects being thrown from above by pedestrians or passing motorists.



FENCES



ELEVATION
PERPENDICULAR TO TRACKS

GRADE SEPARATIONS

GENERAL:

1. FENCING ON THE OVERPASS STRUCTURE SHALL BE REQUIRED, AS SHOWN, ON BOTH SIDES OF EACH STRUCTURE.
2. FENCING, PER PCJPB STANDARD FOR ROW FENCE SHALL BE PROVIDED FOR MINIMUM DISTANCE OF 200 FEET EITHER SIDE OF CENTERLINE OF THE OVERPASS ON BOTH SIDES OF RAILROAD ROW.
3. LIGHTS ARE TO BE INSTALLED ON THE UNDERSIDE OF THE OVERPASS WHERE STRUCTURE SHADOWS WOULD INTERFERE WITH RAILROAD OPERATIONS.
4. CONCRETE SLOPE PAVING SHALL BE REQUIRED ON ALL ABUTMENT SLOPES STEEPER THAN 2 HORIZONTAL TO 1 VERTICAL.
5. PIER PROTECTION IS REQUIRED FOR PIERS WITH LESS THAN 25 FEET HORIZONTAL CLEARANCE TO THE NEAREST TRACK. PIER PROTECTION SHALL BE DESIGNED IN ACCORDANCE WITH THE AREMA MANUAL FOR RAILWAY ENGINEERING, CHAPTER 8, PART 2.1.5 AND THE PCJPB DESIGN GUIDELINES.
6. APPLICANT SHALL BE RESPONSIBLE FOR IDENTIFICATION, LOCATION, PROTECTION AND/OR RELOCATION OF EXISTING UTILITIES. UTILITIES ATTACHED TO THE OVERPASS STRUCTURE MUST BE APPROVED BY PCJPB.
7. SHORING SHALL BE DESIGNED AND APPROVED PER PCJPB GUIDELINE FOR EXCAVATION SUPPORT SYSTEMS.
8. FALSEWORK SHALL BE DESIGNED AND APPROVED PER AREMA, MANUAL OF RAILWAY ENGINEERING CHAPTER 8, PART 28.6.
9. EXCEPTIONS TO THESE STANDARDS MUST BE APPROVED BY PCJPB.

CLEARANCES:

1. PERMANENT VERTICAL AND HORIZONTAL CLEARANCES SHALL BE AS SHOWN.
2. VERTICAL CLEARANCE IS MEASURED FROM THE TOP OF THE HIGHEST RAIL IN FINAL TRACK PROFILE. ADDITIONAL CLEARANCE MAY BE REQUIRED IF SAG VERTICAL CURVE EXISTS OR FOR FUTURE TRACK RAISES.
3. HORIZONTAL CLEARANCE IS MEASURED AT RIGHT ANGLES TO THE CENTERLINE OF TRACK. THE CENTERLINE OF TRACK IS PERPENDICULAR TO THE PLANE OF THE TOP OF RAILS.
4. REQUIRED HORIZONTAL CLEARANCES, AS SHOWN, SHALL BE INCREASED 1-1/2" PER DEGREE OF CURVE WHERE THE STRUCTURE IS LOCATED ON A CURVE OR WITHIN 80 FEET OF THE CURVE LIMITS.
5. MINIMUM TEMPORARY CONSTRUCTION CLEARANCES SHALL BE 21'-6" VERTICAL AND 10 FEET HORIZONTAL. CPUC AUTHORIZATION IS REQUIRED FOR VERTICAL CLEARANCE LESS THAN 22'-6".

FUTURE TRACKS:

1. PROVISIONS MUST BE MADE FOR TWO FUTURE TRACKS AS SHOWN.

DRAINAGE:

1. DRAINAGE FROM THE OVERPASS STRUCTURE SHALL BE DIVERTED AWAY FROM TRACKS AND OTHER RAILROAD FACILITIES.
2. EXISTING DRAINAGE SHALL BE MAINTAINED OR IMPROVED.

FIGURE 3.1 – OVERHEAD GRADE SEPARATION MINIMUM REQUIREMENTS

The overhead shall have a curved fence 8 feet high or straight fence 10 feet high on sides with walkway and a combination of barrier rail and fence with a total height of 10 feet on sides without walkway.

The Deputy Director of Engineering will consider ornamental fencing with a maximum gap of 4 inches and meeting the minimum height requirements above.

The limits of protective fence shall be the full width of the PCJPB Right-of-Way or a minimum of 25 feet beyond the centerline of the outermost track or access road, whichever is greater. Any addition of future tracks may require the lengthening of the safety fences at the expense of the proposing agency.

The limits and types of fences and/or barriers shall be shown on the plans.

The protection and safety of rail operations, train passengers, and PCJPB employees that may be working on the ground beneath the bridge is paramount. The Deputy Director of Engineering must approve any variance to the fence requirements above.

Structures over the PCJPB Right-of-Way shall not have any hinges (especially double hinges) located over the tracks, and they shall be avoided within the PCJPB Right-of- Way.

3.3 Clearances

Clearances will comply with Figure 3.1, with provisions for future tracks, drainage ditches, etc. Permanent clearances are those that will be in place when construction is completed. Temporary construction clearances are those that may be utilized for short periods during the construction of the structure. Any variation of horizontal or vertical clearances will require approval by the Deputy Director of Engineering.

Horizontal clearances are measured at right angles to the centerline of track. The centerline of track is perpendicular to the plane of the top of rails. Vertical clearance is measured from the top of the highest rail in the final track profile.

Special conditions, such as overheads located near passenger stations, may have additional or more restrictive requirements.

3.3.1 Permanent Clearances

Permanent clearances shall be correlated with the methods of construction so that temporary construction clearances will not be less than the minimum allowed.

Overhead bridge structures shall provide the specified horizontal and vertical clearances for anticipated future tracks, changes in track centers, and the raising of tracks for maintenance purposes.

The preferred permanent vertical clearance shall be 25 feet 6 inches above the top of rail for all tracks and at any location under the structure. The minimum vertical clearance shall be 24 feet 6 inches and the absolute minimum vertical clearance shall be 23 feet 6 inches and will only be allowed under special circumstances. Minimum vertical clearance must take into account any superelevation of the rails for tracks on curves. Additional vertical clearances may be required for features beyond those shown in the standard drawing.

The recommended horizontal clearance on tracks shall be 25 feet to the face of pier support. The minimum horizontal clearance on tracks shall be 15 feet to the face of a pier protection wall or column. Required horizontal clearances, as shown in Figure 3.1, shall be increased 1½ inches per degree of curve where the structure is located on a curve or within 80 feet of the curve limits.

The profile of the existing top of rail (1,000 feet each side of the proposed overhead) should be plotted on the plans. If the track profile has a local sag at the proposed bridge location, the vertical clearance from the top of rail to the bridge should be increased to permit raising the track in order to remove the sag.

The proposed minimum permanent vertical and horizontal clearances, as well as the existing clearances of structures to be rehabilitated or replaced, shall be indicated on the drawings.

Vertical and horizontal clearances shall be adjusted so that the sight distance to railroad signals is not reduced, unless the signals are to be relocated as part of the project.

Horizontal clearances may need to be modified to include access roads, where such roads now exist or the railroad ROW has sufficient width for new roads.

In any case, clearances shall not be less than those required by California Public Utilities Commission (CPUC) General Order 26-D.

3.3.2 Temporary Construction Clearances

The proposed minimum temporary vertical and horizontal construction clearances known to be required during design shall be indicated on the drawings. If temporary construction clearances are found to be required during construction, the proposed clearances shall be submitted to PCJPB for approval.

Minimum temporary vertical clearance shall be 21 feet 6 inches above the top of rail for all tracks and at any location under the structure. Minimum temporary vertical clearance must take into account any superelevation of the rails for tracks on curves. For proposed vertical clearance less than 22’-6 inches, the proposing agency will be required to obtain CPUC approval.

Minimum temporary horizontal side clearance from centerline of track shall be 10 feet. This distance shall be increased 1½ inches per degree of curve where the temporary clearance restriction is located on a curve or within 80 feet of the curve limits.

Construction material or equipment shall not be placed less than 15 feet from the centerline of the nearest track without permission of PCJPB.

3.4 Special Provisions

3.4.1 Drainage and Erosion Control

Maintaining the existing drainage and providing for future drainage improvements is a major concern for PCJPB. Existing track ditches must be maintained at all times.

Drainage plans shall be included with the documents submitted for review. If the proposed project will not change the quantity and/or character of flow in the railroad’s ditches and/or drainage structures, the submitted documents shall include a general note stating this. If an increase in current drainage requirements is required, the plans must include hydrologic computations indicating the rainfall intensity and duration of the design storm used and the method of analysis. Where project design calls for the drainage flow to increase through the railroad right-of-way, methods must be developed to carry the additional flow. Drainage ditches or structures shall be designed for a 100-year storm event such that the water surface elevation does not exceed the top of subgrade elevation of the track.

In order to evaluate the impact of the proposed project relative to the existing site drainage, cross sections perpendicular to the centerline of track should be submitted along with the drainage plans. Sufficient cross sections should be submitted to adequately depict the site conditions. One cross section is to be taken along the centerline of the overhead. The existing railroad ditch and the proposed toe of slope for the approach fill should be located on all cross sections.

Approval of the drainage plan does not relieve the submitting agency and/or designer of ultimate responsibility and liability for a satisfactory drainage design.

No scuppers or other deck drains, roadway drainage, catch basins, or other drainage features required by the project are permitted to drain onto PCJPB property. Any variance of this policy must have the approval of the Deputy Director of Engineering. If a variance is granted, deck drains and scuppers will not be permitted to discharge water onto the track or access roads. Downspouts attached to the substructure shall be used to convey the water to a storm drain system or the drainage ditches. Erosion protection, such as aggregate or splash blocks must be provided at outlets into ditches. Downspouts shall not be located on the face of piers nearest the track(s).

If any drainage must be conveyed into a railroad ditch, calculations that indicate the ability of the ditch to carry the additional runoff must be provided to the PCJPB for approval.

Abutment or approach slopes must include methods to control erosion and prevent material from sloughing into drainage ditches or onto the track. Slopes shall not be steeper than 1.5 horizontal to 1 vertical.

If deck drainage or highway drainage is to be discharged onto the embankment slopes, concrete slope paving must be used. Concrete slope paving is required for slopes steeper than 2 horizontal to 1 vertical. Slope paving shall extend for a minimum of 2 feet beyond the outside edges of the bridge. Where warranted, the slope paving shall be extended around the ends of the abutment to a line parallel with the tracks. Slope paving shall consist of a prepared subbase and filter fabric with a minimum of 4-inch-thick reinforced concrete placed on prepared sub-base and filter fabric. The toe of slope paving shall terminate at the bottom of the drainage ditch.

The plans shall show all permanent erosion control methods. Temporary erosion control during construction shall be submitted to the PCJPB for approval prior to beginning any grading work.

3.4.2 Utilities

The applicant shall be responsible for identification, location, protection and/or relocation of existing utilities. Utilities to be attached to the overhead structure must be approved by PCJPB. New or modified existing utility crossings or encroachments on PCJPB Right-of-Way must comply with applicable CPUC General Orders and PCJPB Standards and be approved by PCJPB.

The plans shall include identification and location of all existing and proposed utilities. The plans shall indicate parties responsible for installation or modification of utilities.

Approval by the Deputy Director of Engineering is required if changes to PCJPB signal, communication or other utility facilities are proposed.

Fiber Optic cables affected by the project will be relocated at the project's expense.

3.4.3 Lighting

Permanent lighting of the track area shall be provided for new or modified overhead structures exceeding 80 feet of superstructure width. Lighting shall also be provided for structures having widths of less than 80 feet in areas where switching is performed, within 100 feet of station platforms or where high vandalism or trespassing has been experienced.

Temporary lighting shall be provided for falsework, regardless of the superstructure width, in areas where switching is performed, within 100 feet of station platforms or where high vandalism or trespassing has been experienced.

Lighting shall, as a minimum, maintain an average of one foot-candle for the area under the structure at the PCJPB tracks. Fixtures shall be installed without reducing the minimum clearances.

Maintenance of lights shall be the responsibility of the agency. Access to perform any maintenance for lights shall be coordinated with the PCJPB.

Structures with separation greater than 10 feet from each other shall be considered as independent structures for the purposes of lighting.

3.4.4 Railroad Electrification

PCJPB/Caltrain is currently under construction for electrification of the corridor. All structures shall provide for a future overhead catenary system. Space for catenary support poles shall be provided for along the PCJPB Right-of- Way and on structures. Overhead, vertical, clearance and horizontal clearance shall comply with the following references: CPUC GO 26-D and GO95, NESC, NEC, AREMA Chapters 28 and 33, and CPUC Resolution SED-2 dated November 10, 2016.

All concrete superstructures and substructures shall be detailed to mitigate the effects of stray current corrosion of steel reinforcing, prestressing elements, and other steel components. This will require that electrical continuity be provided between all steel elements within each concrete structural component and be run to a central location at time the structure is designed. When the corridor is electrified the central location points will be connected to the corrosion control system. Comply with GO95, AREMA Chapter 33, IEEE, NESC, and NEC provisions for stray current.

3.5 Substructure

Wherever practical, overhead bridge structures shall have all piers and abutments located outside of the railroad right-of-way.

Footings for all piers, columns, walls or other facilities shall be located and designed so that any temporary sheeting and shoring for support of adjacent track or tracks during construction will not be closer than 8 feet 6 inches from centerline of track. Excavations will not be allowed closer than 8 feet 6 inches from centerline of track unless specifically approved by the PCJPB.

Pier footings tops within 25 feet of the nearest track centerline shall be a minimum of 6 feet below base of rail. They should not restrict PCJPB from modifying longitudinal drainage systems in the future or from providing unobstructed areas for placing, signal, fiber optic lines or other buried utilities.

The potential for railroad live load on foundations should be investigated when determining pier locations. Drilling of shafts or shoring construction for footings within the influence of track surcharge shall not proceed without the approval from the PCJPB. For limits of track surcharge influence refer to PCJPB Engineering Standards for Excavation Support Systems, Railroad Zone of Influence diagram.

Pier protection walls or “crash walls” are required when a face of pier is closer than 25 feet from the centerline of track. Pier protection walls shall comply with the American Railway Engineering and Maintenance of Way Association (AREMA) Manual of Railway Engineering, Chapter 8, Part 2, Section 2.1.5. Pier protection walls

may be omitted if the piers are shown to be of HEAVY construction. HEAVY construction is defined as piers having a cross-sectional area equal to or greater than that required for the pier protection wall and the larger of its dimensions is parallel to the track.

Piers, abutments, retaining walls or other structures shall be located so that they do not interfere with drainage requirements.

Piers should be located with their primary axis parallel to the track.

3.6 Construction

The contractor must enter into a right of entry agreement with PCJPB prior to entering PCJPB property. Required insurance must be in effect for the duration of the project or as required in the agreement.

Construction schedules, work sequences and erection plans shall be provided to PCJPB for review and approval. If existing structures are to be removed, demolition plans and procedures shall be submitted to PCJPB for review and approval. Approvals by PCJPB must be obtained before any work begins.

At grade crossings of PCJPB tracks will not be allowed unless approved by the PCJB.

Falsework required for the project must meet or exceed the minimum temporary clearance requirements. Falsework shall comply with the requirements of the AREMA Manual for Railway Engineering Chapter 8, Part 28.6. Falsework plans and design calculations shall be stamped by a Registered Professional Engineer and shall be submitted to PCJPB for review and approval.

Shoring and trenching shall be designed and approved in accordance with the PCJPB Engineering Standards for Excavation Support Systems. Shoring for excavations will be required if excavations occur within the “Zone of Influence” as defined in the PCJPB Engineering Standards for Excavation Support Systems. Shoring shall be designed for the track dead load and Cooper E-80 railroad live load. Shoring plans and design calculations shall be prepared and stamped by a Registered Professional Engineer and shall be submitted to PCJPB for review and approval.

Shoring, trenching, pits, etc., within 15 feet of centerline of track shall be protected by guardrails.

Temporary construction clearances less than those allowed in CPUC General Order 26-D

require approval by PCJPB and CPUC. Approvals are the responsibility of the public agency sponsoring the project.

Safety rail or barriers placed parallel to the tracks may be required to inhibit or prevent accidental incursion into the railroad operating envelope.

3.7 Miscellaneous

Walkways shall be provided adjacent to turnouts and tracks where train personnel are required to work on the ground. Walkways shall conform to CPUC General Order 118.

Visibility of railroad signals must not be restricted or obscured at any time.

PCJPB Right-of-Way shall be fenced for a minimum of 200 feet on each side of the overhead.

All primary dimensions in drawings for overhead grade separation structures are to be shown in English units. Projects that require the use of metric units shall indicate all dimensions and design criteria assumptions in dual units on the project general plan sheet. English units are to be shown in parentheses. Primary dimensions refer to but are not limited to the following:

a) Horizontal and vertical clearances.

b) Track spacing, track stationing, and railroad right-of-way.

c) Top of rail elevation under structure and grade profile.

d) Span length, width and depth of superstructure elements.

e) Size and limits for barriers and fences.

f) Location and elevation of underground or aerial utilities and their relocation adjustments if required.

g) Size, elevation and location of pier or abutment footings for spans adjacent to railroad tracks.

h) Size of structure supports (pier or abutment walls, columns).

i) Size and elevations of pier protection walls, if required.

j) Shoring location and limits, if required.

k) Size and location of drainage structures and ditches.

l) Temporary construction vertical or horizontal clearances, if required.

CHAPTER 4

SEISMIC DESIGN

CHAPTER 4: SEISMIC DESIGN

The goal of these criteria is to minimize safety risk and to achieve a projected post-earthquake performance that is consistent with the function and importance of a JPB's facility or equipment. For life safety consideration, the goal is to design all structures to a noncollapse seismic performance for an extreme level seismic event. For functional and operational consideration, the goal is to avoid prolonged interruption to operations and to protect the capital investment for lower-level functional seismic events.

All structures, equipment, utilities, and related supports and anchorages shall be designed to resist the ground motions and meet the acceptance criteria specified in this document. All components shall meet, as a minimum, the provisions in applicable State and National codes, which are generally focused toward life safety. Applicable State and National codes are used as much as possible when applicable. In the case of conflict among the referenced codes, the applicable referenced code with the most stringent requirements governs.

These Seismic Criteria make reference to, or incorporate (with or without modification) the following principal design codes:

- AREMA – American Railway Engineering and Maintenance of Way Association, Manual for Railway Engineering
- ASCE/SEI 7 – American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures
- AASHTO LRFD Bridge Design Specifications – 8th Edition
- CBC – California Building Code
- Caltrans Seismic Design Criteria, Version 2.0 (CSDC)
- Caltrans Seismic Design Specifications for Steel Bridges, 2016
- Caltrans Amendments To AASHTO LRFD Bridge Design Specifications – 8th Edition, 2019

Typical JPB structures are categorized into the following structural types:

1. Buildings – Aboveground station and facility buildings, and other nonbuilding facility structures
2. Equipment and Nonstructural Components – Nonstructural components and all equipment, devices, utilities, and their associated supports and anchorages
3. Earth-Retaining Structures – U-walls and retaining walls
4. Underground and Other Structures – Tunnels, underground stations, and other structures not specified elsewhere

5. Bridges – Bridges and other structures carrying trains

4.1 Buildings

For the Buildings structural type, seismic design shall be governed by CBC and ASCE7. K-Braced Frames are not allowed as Ordinary Concentrically Braced Frames. Tension-only bracing members in the Ordinary Concentrically Braced Frames are not allowed for buildings designated as essential by JPB, which are required to remain operational for train operations after a functional seismic event. Essential buildings shall be designed for I = 1.5 and Occupancy Category IV, in accordance with CBC.

4.2 Equipment and Nonstructural Components

For the Equipment and Nonstructural Components structural type, seismic design shall comply with the provisions of CBC Section 1613, Earthquake Loads, and ASCE/SEI 7, Chapter 13, Seismic Design Requirements. Equipment and nonstructural components designed by JPB as essential shall be designed using an Importance Factor of Ip = 1.5. Equipment, components, and utilities that are required to support normal train operations after a functional level seismic event are considered essential. These include fire protection; emergency power/lighting; sump discharge piping systems; UPS; batteries; inverters; power and control equipment; train control/ equipment, including trackside switch machines and trackside detection equipment; traction power; and auxiliary power equipment. Nonessential equipment and nonstructural components shall be designed using an Importance Factor of Ip = 1.0.

4.3 Earth-Retaining Structures

For the Earth-Retaining Structures structural type, structures shall be designed in accordance with AREMA except that seismic soil pressures shall be determined based on AASHTO LRFD Bridge Design Specifications, Article 11.6.5 – Seismic Design for Conventional Retaining Walls with 2019 Caltrans Amendments.

4.4 Underground and Other Structures

For the Underground and Other Structures structural type, project-specific seismic design criteria shall be developed. Project-specific seismic design criteria may also be required for nonstandard or important structures, and for large-scale programs if determined necessary by JPB. Project-specific seismic design criteria shall be reviewed by a peer review panel retained by the designer. The review panel, as a minimum, shall include a California-registered Geologist/Geotechnical Engineer with a minimum of 10 years of practice in seismic ground motions generation, and a California registered Civil/Structural Engineer with a minimum of 10 years of practice in seismic structural design. Members of the review panel are subject to JPB’s review and approval.

4.5 Bridges

For the Bridges structural type, seismic design shall be in accordance with AREMA Chapter 9 and CSDC as supplements. Additional modifications are also made to eliminate unnecessary design efforts and inconsistency between AREMA and CSDC, and to adjust for the difference in performance priority between freight and commuter rail service. A summary of the relevant aspects of AREMA Seismic Design Criteria and CSDC is included in Sections 4.5.1 and 4.5.2, respectively, for reference. Seismic design criteria for JPB bridges are described in Section 4.5.3.

4.5.1 AREMA Seismic Design Criteria

AREMA recommends the use of a three-level ground motion and performance criteria approach to be consistent with its recommended railroad post-seismic event response procedures. Earthquakes are extreme events associated with a great amount of uncertainty and risk factors that are an integral part of seismic design. To achieve a balance between seismic risk and costs associated with risk reduction, a certain amount of risk must be accepted.

The greatest amount of uncertainty is associated with the seismic hazard at the site. Therefore, the overall seismic risk of a bridge is strongly affected by the design ground motions used. The three levels of seismic events defined in AREMA seismic design criteria are:

- Level 1 Ground Motion, which represents an occasional event with a reasonable probability of being exceeded during the life of the structure
- Level 2 Ground Motion, which represents a rare event with a low probability of being exceeded during the life of the structure
- Level 3 Ground Motion, which represents a very rare or maximum credible event with a very low probability of being exceeded during the life of the structure

The acceptable risk criteria with respect to Level 1 Ground Motion shall consider the safety and continuing operation of trains with speed restrictions. For Ground Motion Levels 2 and 3, the acceptable risk criteria may be based mainly on economic considerations, unless the bridge has a high passenger train occupancy rate. Additionally, train traffic is stopped for Ground Motions Levels 2 and 3 until bridge inspections are completed in accordance with railroad post-seismic event response guidelines. AREMA further recommends the following specific performance criteria for each of the three seismic event levels.

1. Serviceability – to minimize the probability of affecting the safety of trains at restricted speeds due to an occasional seismic event (AREMA Level 1 Ground Motion). The Serviceability limit state contains restrictions on bridge stresses, deformations, vibrations, and track misalignments. Critical members shall remain in the elastic range. Only moderate damage to the structure is allowed. The structure shall not suffer any permanent deformation due to nonelastic deformations or liquefaction of the foundation soil.

2. Ultimate – to minimize the probability of repair downtime and costs due to a rare seismic event (AREMA Level 2 Ground Motion). The Ultimate limit state requires that the overall structural integrity is not compromised. The strength and stability of critical members shall not be exceeded. The structure may respond beyond the elastic range, but displacement, ductility, and detailing requirements shall be satisfied to reduce damage and loss of structure use. The damage should occur as intended in design and be readily detectable and accessible for repair. The structure shall not suffer any damage that threatens the overall integrity of the bridge due to deformations or liquefaction of the foundation soil.

3. Survivability – to maximize the probability of the structural survival due to a very rare seismic event (AREMA Level 3 Ground Motion). The Survivability limit state requires the structural survival of the bridge. Extensive structural damage, short of bridge collapse, may be allowed. Structural and geometric safety measures that add redundancy and ductility shall be used to reduce the likelihood of bridge collapse. Failures of the foundation

soil shall not cause major changes in the geometry of the bridge. Depending on the importance and the replacement value of a bridge, an individual railroad may allow irreparable damage for the Survivability limit state, and opt for new construction.

It should be noted that even though AREMA's design criteria allow certain levels of structural damages for the Ultimate and Survivability states, there is no analysis procedure to calculate structural responses beyond the elastic limit. In addition, there are quantitative criteria to evaluate potential damage to structural members that are projected to experience demand forces beyond the elastic limits.

Supplemental design criteria are necessary to address this need.

4.5.2 Caltrans Seismic Design Criteria

CSDC employs different design performance criteria for three different categories of bridges: Ordinary, Recovery, and Important. Important Bridges are those expected to provide immediate access to emergency and similar life-safety facilities after an earthquake, those whose closure would create a major economic impact, or those formally designated as critical by a local emergency management plan. Recovery Bridges serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake. Ordinary Bridges are any not designated as either Important or Recovery.

For an Ordinary Bridge, a single-level seismic evaluation is used for seismic design. The performance goal of an Ordinary Bridge is to minimize the probability of bridge collapse during a Safety Evaluation Earthquake (SEE), while allowing for major damages that would require bridge replacement.

For a Recovery Bridge, a two-level seismic evaluation is used for seismic design. The two-level performance goals of a Recovery Bridge are:

1. To minimize the probability of bridge repair requiring extensive downtime due to a Functional Evaluation Earthquake (FEE), while allowing for moderate damages

2. To minimize the probability of bridge replacement due to an SEE, while allowing for minor damages

For an Important Bridge, a two-level seismic evaluation is used for seismic design. The two-level performance goals of an Important Bridge are:

1. To minimize the probability of any service downtime due to an FEE, while allowing for only minimal damages

2. To minimize the probability of bridge repair need requiring minor downtime due to an SEE, while allowing for moderate damages

CSDC also specifies comprehensive procedures and requirements pertaining to seismic hazards, seismic design philosophy, analysis methods, deformation demands and capacities, and seismic detailing. These design criteria elements address the shortcomings of AREMA design criteria.

4.5.3 JPB Seismic Design Criteria

JPB Seismic design criteria follow the framework and guidelines specified in AREMA Chapter 9, Seismic Design for Railway Structures, with modifications to simplify the method used to determine the ground motion level for each of the three-level performance criteria. Because AREMA does not provide analysis procedures to calculate structural responses beyond the elastic limit, nor quantitative criteria to evaluate damages of structural members that are subjected to seismic responses beyond the elastic limits, CSDC is used to supplement these deficiencies.

Performance Criteria

JPB bridges shall be designed to meet the three-level earthquake performance goals, as shown in Table 4.1.

TABLE 4.1 – SEISMIC PERFORMANCE CRITERIA

Performance Goal	Seismic Hazard Level	Expected Post-Earthquake State	Expected Post-Earthquake Service Level
Serviceability	L1	No damage to minimal damage	Full service
Ultimate	L2	Minimal to moderate damage	Service after repair
Survivability	L3	Major damage without collapse	No service

The primary aim of the Serviceability limit state is to ensure the safety of trains. After Level 1 earthquakes, trains are allowed to proceed at a reduced speed until inspections are completed and the track is cleared. The stresses and deformations are limited to the immediate use of the structure with restricted speed. Even though structures are expected to remain in elastic response range, vibration of flexible bridges with natural periods in the transverse direction around 1 second may cause derailments. However, JPB railroad bridges are typically short, simple span bridges, with relatively regular configurations, and are traversed with a continuous track structure; historically, they have performed well in seismic events. It is believed that, in the large majority of cases, bridge design will likely be governed by load combinations including centrifugal loads, wind loads, and lateral loads from heavy equipment, other than the load combinations that include seismic load under Serviceability-level earthquake. Additionally, test results from the Strawberry Park Underpass, a steel-through-girder ballast deck bridge in Los Angeles, have demonstrated that railroad structures may have significant lateral restraint capacity and continuity, although further research and testing needs to be performed to quantify that capacity. Therefore, the Serviceability seismic performance requirement is most likely met for a regular JPB bridge, the majority of the PCJPB bridge inventory, when properly designed for service loads. If a quick comparison of service loads and seismic demands verifies that service loads control the design, additional design calculations are not required to minimize design efforts unless directed otherwise by JPB Engineering.

The primary aim of the Ultimate limit state is to minimize the extent of damage and to ensure the overall structural integrity of the bridge. After Level 2 earthquakes, trains are stopped until inspections are completed. Structural damages that can be readily detected and economically repaired may be allowed. Structures should be repairable within few weeks to support a limited level of train service, and ideally be repairable within a few months to allow for normal train operations. By allowing structures to respond beyond the elastic range and undergo inelastic deformations, the earthquake resistance capacity of a bridge with good ductility is significantly increased.

The Survivability limit state aims to prevent overall bridge collapse. After Level 3 earthquakes, the expected track damage would prevent immediate access to the bridge. Extensive irreparable structural damages, short of bridge collapse, may be allowed. Bridges are likely to be replaced after this level of earthquakes. The performance of a bridge during such earthquakes will mainly depend on the ductility and redundancy characteristics of the bridge and on the additional safety measures designed to prevent bridge collapse.

Ground Motion Levels

Design seismic hazards are typically characterized by the ground motion levels of a site. As specified in AREMA, the three design ground motion levels are defined in terms of peak ground acceleration and acceleration spectrum associated with a given average return period range, as shown in Table 4.2.

TABLE 4.2 – GROUND MOTION LEVELS

Ground Motion	Level	Frequency Average Return Period (Yrs.)
1	Occasional	50 to 100
2	Rare	200 to 475
3	Very Rare	1,000 to 2,475

Additionally, AREMA employs a specific methodology to determine the return period for each ground motion level, based on the level of seismic risk in life safety and economics. The return period for each limit state of a bridge is calculated using a set of three structure importance measures and the associated weighting factors for that limit state.

The three structure importance measures are Immediate Safety, Immediate Value, and Replacement Value. Immediate Safety is a measure of the magnitude of earthquake a structure should be able to survive without any interruption of service. This measure is determined based on occupancy, hazardous material, and community life line considerations. Immediate Value is a measure of the magnitude of earthquake a structure should be able to survive with an interruption of service but with the ability to return to service after minor repairs. This measure is determined based on the railroad's use of the structure and the ability to detour around the structure. Replacement Value is a measure of the magnitude of the Ultimate earthquake the structure should be able to survive. This measure is determined based on the difficulty of replacing the structure according to the bridge geometry. The following values shall be used for these three measures to design JPB Bridges:

- Immediate Safety = 4, due to high volume of commuter train service

- Immediate Value = 4, due to high volume of commuter train service and lack of a detour route
- Replacement Value = 3, conservative for a bridge span less than 125 feet, a bridge length less than 1,000 feet and a bridge height less than 40 feet

To calculate the importance classification factor for each limit state, add the Immediate Safety, Immediate Value, and Replacement Value measures together after multiplying them by the appropriate weighting factors shown in Table 4.3.

TABLE 4.3 – WEIGHTING FACTORS

Limit State	Immediate Safety	Immediate Value	Replacement Value
Serviceability	0.80	0.20	0.00
Ultimate	0.10	0.80	0.10
Survivability	0.00	0.20	0.80

The calculated importance classification factors for the three design limit states are:

- Serviceability = 4 (4 × 0.8 + 4 ×0.2 + 3 ×0.00)
- Ultimate = 3.9 (4 ×0.1 + 4 ×0.8 + 3 ×0.10)
- Survivability = 3.2 (4 ×0.0 + 4 ×0.2 + 3 ×0.8)

The return period for each limit state shall be calculated using a linear relationship between the appropriate average return period limits shown in Table 4.2. To calculate the return period for each limit state, multiply the importance classification factor by the difference between the maximum and minimum return periods and divide by 4; add this result to the minimum return period to get the final value. The calculated return periods are:

- Serviceability Level events, return period = 100 years
- Ultimate Level events, return period = 468 years
- Survivability Level events, return period = 2,180 years

Survivability-level earthquakes are extreme events associated with a great amount of uncertainty, from many sources. The greatest source of uncertainty is associated with the regional seismicity and the expected ground motion characteristics at the site. In the San Francisco Bay Area, the calculated return period of 2,180 years for a Survivability event represents an extremely high level of seismic hazard. This seismic hazard level is much higher than the safety-level earthquakes used in CSDC and BART Seismic Design Criteria, both of which are based on return periods of around 1,000 years. Designing a bridge for such a high level of ground motion is economically undesirable, unless there is a severe social penalty associated with bridge failure. A reasonable amount of risk must be accepted so that a balance between the probability of extreme earthquakes exceeding the design event and the costs of overdesign can be achieved. In addition, train traffic will be stopped for structure inspections after such an extreme event, in accordance with JPB's post-seismic response procedure. Safety considerations are properly addressed. Therefore, the return period for a Survivability earthquake is adjusted to 1,000 years, rather than the 2,180 years calculated above, for designing JPB Bridges.

With the return period defined, base acceleration coefficients to define the seismic design spectra may be determined by using the seismic hazard maps and site-specific data published on the United States Geological Survey (USGS) website. USGS manages the United States National Seismic Hazard Model, which defines the potential for earthquake ground shaking for various probability levels across the conterminous United States and is applied in seismic provisions of building codes, insurance rate structures, risk assessments, and other public policy. The model represents an assessment of the best available science in earthquake hazards and incorporates new findings on earthquake ground shaking, seismicity, and long-period amplification over deep sedimentary basins. The output from the National Seismic Hazard Model is a suite of seismic hazard curves calculated on a grid of latitude/longitude locations that describe the annual frequency of exceeding a set of ground motions. The output includes hazard curves and probabilistic hazard data and maps for VS30 equal to 760 meters per second (m/s) and 260 m/s (NEHRP site class B/C and D), for 0.2-, 1.0-, and 5.0-second periods, as well as peak ground acceleration. The maps depict probabilistic ground motions with a 2 percent, 5 percent, and 10 percent probability of exceedance in 50 years. Spectral accelerations are calculated for 5 percent damped linear elastic oscillators. Base acceleration coefficients with return periods not given may be determined based on formulas in AREMA.

Seismic Design Philosophy

Seismic design of JPB Bridges should maintain a balance between functional requirements, cost, and seismic-resisting features. Provisions in AREMA Chapter 9, Article 1.4.3, Conceptual Design, should be followed as closely as is practical. Special considerations shall be given to the preferred bridge configurations and superstructure characteristics, and any potential ground stability issues shall be addressed.

JPB Bridges shall be designed based on “strong beam – weak column” proportioning principle. Plastic hinging shall be limited to Seismic Critical Members (SCMs), allowing a mechanism to form to facilitate transverse and longitudinal movement of bridge bents and frames. The plastic hinge ductility or other means of energy dissipation/bridge damping shall be adequate to satisfy the deformation demands imposed by the design seismic hazards. Every bridge shall be designed with an Earthquake-Resisting System that ensures a load path for gravity loads and provides sufficient strength and ductility to achieve the performance criteria specified in CSDC, Section 1.3.

Seismic Analysis Methods and Demands

Serviceability limit state evaluations, if required, use elastic analysis methods to determine stresses and deformations as specified in AREMA Chapter 9, Article 1.4.5, Analysis Procedures. The methods recommended include (1) the Equivalent Lateral Force Procedure, which is applicable to regular bridges; and (2) the Modal Analysis Procedure for multi-span irregular bridges.

For Ultimate and Survivability design, structural responses beyond elastic limits are expected to occur. Seismic deformation demands, including displacements, rotations, curvatures, or strain, shall be determined in accordance with CSDC, Section 4, Seismic Deformation Demands and Analysis Methods.

Where applicable, the effects of continuous welded rail across a structure shall be considered in the evaluation of the structure.

Horizontal displacements due to train loads may be neglected and need not be combined with seismic displacement. The train loads shall not be considered as contributing to the system's “dynamic” mass for the purpose of equivalent static or dynamic seismic analysis.

Train loads generally shall not be considered for a seismic analysis for a typical bridge. Where applicable (e.g., on long viaducts or when specific analysis methods require that train loads be included), train loads may be modeled as equivalent distributed loads. Where equivalent distributed loads are used in the analysis, the designer shall account for any local or global effects to the structure due to actual concentrated axle loads.

Seismic Deformation Capacities and Response Limits

For the Serviceability limit state design, seismic response in stresses and deformations shall be within the elastic range. Structures shall be designed in accordance with AREMA.

For Ultimate and Survivability design, structural responses beyond the elastic limits are expected to occur. Deformation capacities beyond the elastic limits need to be determined for seismic design. Seismic deformation capacities of bridge members, frames, and bents shall be determined in accordance with CSDC, Section 5, Seismic Deformation Capacities. Inelastic Static Analysis (ISA) shall be used to determine the displacement capacity, Δ_C, of a frame or bent, provided that the seismic response of the structure is dominated by a single translational mode of vibration. Δ_C shall correspond to the lateral capacity of the bridge or bent when any plastic hinge reaches its ultimate curvature capacity, ϕ_u. ISA shall be performed with the dead load applied while a lateral static load or displacement is applied at the center of gravity of the superstructure or the bent cap. Nonlinear soil-foundation-structure interaction shall be taken into account by means of soil springs. Abutment stiffness shall be included. For simple bridges where Equivalent Static Analysis (ESA) is used to determine displacement demand, local displacement capacity equations may be used as the ISA.

Global displacement of each frame or bent beyond elastic limits shall satisfy:

- Survivability limit state – $\Delta_C \geq \Delta_D$
- Ultimate limit state – $\Delta_C \geq 1.4\Delta_D$

where:

Δ_C = frame or bent displacement capacity in the local principal/critical axis of member
Δ_D = frame or bent displacement demand in the local principal/critical axis of member
Δ_D and Δ_C shall be measured in the same local principal/critical axis.

In addition, seismic critical members (SCM) as defined in CSDC shall be designed with adequate ductility. Displacement ductility demand, μ_D, shall be used to measure the ductility of a SCM, which shall be defined as $\mu_D = \Delta_D / \Delta_{Y(i)}$.

where:

Δ_D = frame or bent displacement demand in the local principal/critical axes of a member
$\Delta_{Y(i)}$ = frame or bent displacement at the instant a plastic hinge forms in the i-th SCM

The displacement ductility demand values for SCMs shall not exceed the values specified in Table 4.4.

TABLE 4.4 – DISPLACEMENT DUCTILITY DEMAND LIMIT, M_D

Plastic Hinge Location	Seismic Critical Member	Survivability Limit State	Ultimate Limit State
Substructure	Column in a single-column bent supported on a footing or Type II shaft	4.0	2.5
	Column in a multi-column bent supported on a footing or Type II shaft	5.0	3.5
	Pile extensions or Type I shaft, plastic hinge at cap/superstructure soffit	5.0	3.5
Foundation	Pile extensions or Type I shaft, plastic hinge below ground	3.5	2.5
	Pile groups in Class S2 soil, fixed pile to cap connection, plastic hinge on top of pile	2.5	1.5
	Pile groups in Class S2 soil, pinned pile to cap connection	1.0	1.0
	Pile groups in Class S1 soil	*	*

Notes:

* Lateral analysis not required for foundations in Class S1 soil. See CSDC, Section C6.2.3.1.

** Refer to CSDC for definitions of substructure type, foundation types, and soil class.

Capacity-protected members such as superstructures and bent caps shall be designed to resist the over-strength demands imparted by SCMs and sacrificial elements, in accordance with CSDC, Section 4.4.3.

The effects of gravity loads acting through lateral displacements shall be included in the design.

For SCMs meeting the ductility demand limits specified in Table 4.4, P-Δ effects may be ignored if P-Δ moment due to dead load is less than 25 percent of the idealized plastic moment capacity of a SCM. Refer to additional details in CSDC, Section 4.4.4.

Foundations, Abutments, and Soil-Foundation-Structure Interaction

Foundation soils combined with the structural components and the seismic input loading determine the dynamic response of the foundation subsystem. Bridge foundations, including piles, shafts, and footings (pile cap and spread footings) shall be designed to resist seismic loading to meet the seismic performance criteria specified above and in CSDC, Section 6.2, Foundations. Foundation components classified as seismic critical members or capacity-protected members shall satisfy all applicable requirements of the seismic design criteria herein.

Typically, the soil response has a significant effect on the overall foundation system response. Therefore, the anticipated foundation subsystem response can be based on the characteristics of the soil within the foundation's zone of influence. CSDC classifies site soil profiles into Class S1 and Class S2. Class S1 represents competent soils and Class S2 represents noncompetent soils, including marginal soil, poor soil, soft soil, potentially liquefiable soil, and soils susceptible to lateral spreading. Refer to CSDC, Section 6.1 for detailed definitions of these soil classes.

Spread footings design shall include flexure, one-way shear, and two-way shear. Flexural and shear demands shall be based on the axial load, column over-strength moment, and the associated shear. Footing-to-column moment-resisting joints shall be proportioned to satisfy joint shear requirements.

Pile foundation systems shall be designed to resist the demands imposed by the over-strength moment and shear force of the columns, and the lateral displacement of the foundation considering soil-foundation-structure interaction, except for piles founded in Class S1 with simple configurations. The axial force demand shall not exceed the factored nominal seismic resistance provided by the geo-professional. The piles shall maintain their axial load capacity at the expected lateral displacement. If the deformation demand creates plastic hinging in the piles, the piles shall be designed as seismic critical members.

All shafts shall be constructed with diameters equal to or greater than the maximum dimension of the supported column. Type I shafts shall be designed so that the cross section of the confined core is the same for both the column and the shaft, but the concrete cover and area of transverse and longitudinal reinforcement may change between the column and the shaft. Type II shafts shall be at least 24 inches larger than the maximum dimension of the supported column. Shafts supporting columns by means of a pin between the column base and the top of the shaft shall be designed as capacity-protected members based on the over-strength shear and moment (if any) demands at the base of the column. If a reduced-diameter reinforcement cage is used to form the pinned connection, the pin rebar cage shall be developed in both the column and the shaft. If a pipe or solid steel section is used to form the pinned connection, the lower portion of the pipe/ solid section shall be developed in the shaft.

The backfill passive pressure force-resisting movement at the abutment varies nonlinearly with longitudinal abutment displacement and is dependent on the material properties of the backfill. Although the full nonlinear abutment backbone curve or the bilinear representation of the backbone curve may readily be used with nonlinear time history analysis, the bilinear representation of the backbone curve is most suited for elastic static, elastic dynamic, and inelastic static analyses. Refer to CSDC to determine the effective longitudinal stiffness to be used for various types of seismic analysis.

A nominal transverse spring stiffness equal to 50 percent of the elastic transverse stiffness of the adjacent bent may be used at the abutment in the elastic demand assessment models. Any additional element such as shafts (used for transverse ductility) shall be included in the transverse analysis with a characteristic force-deflection curve. The initial slope of the force-deflection curve shall be included in the elastic demand assessment model. Transverse stiffness of diaphragm type abutments supported on standard piles surrounded by dense or hard material may conservatively be estimated, ignoring the wingwalls, as 40 kips per inch per pile.

The support length normal to the centerline of the backwall shall satisfy the requirements in CSDC, Section 6.3.3, Support Length. The shear key force demand for abutments supported on piles and spread footings shall be determined in accordance with CSDC, Section 6.3.4, Shear Key Design.

Seismic Detailing and Other Provisions

Comply with seismic requirements for splices, transverse reinforcement, and development of main flexural/longitudinal reinforcement steel in CSDC, Section 8, Seismic Detailing. Appropriate detailing provisions in AREMA Chapter 9, Article 1.4.7, Detailing Provisions shall be incorporated into the structure design to meet the performance requirements for Ultimate and Survivability limit states.

CHAPTER 5

PEDESTRIAN UNDERPASS

CHAPTER 5: PEDESTRIAN UNDERPASS

Purpose

This guideline was created to provide standards and requirements regarding design and construction of new or modified pedestrian underpass structures that affect the tracks and property of the Peninsula Corridor Joint Powers Board (PCJPB). Design engineers should use these guidelines as the basis for preliminary and final design.

5.1 General Requirements for Design of Underpasses

Design of pedestrian underpasses shall comply with the appropriate parts of the current AREMA Manual for Railway Engineering. Applicable sections of the manual include Chapter 8, Part 2, 5, 16, and Chapter 1, Part 4 among others.

A minimum of 3 feet depth shall be maintained between the bottom of the track tie to the top of the pedestrian underpass.

The minimum compressive concrete strength used for underpasses shall be 3,600 psi at 28 days.

Reinforcement for pedestrian underpasses or box shall meet the following requirements:

- ASTM Standard A615 Grade 60, or
- ASTM Standard A706, or
- Welded steel wire fabric ASTM Standard A497

A maximum allowable steel tensile stress of 24,000 psi shall be used in service load design.

5.1.1 Design Considerations

The selection and design of underpasses shall be based on:

- The purpose of the structure (pedestrian, equipment access, etc.)
- Stresses and loads to be applied to the top and invert level
- Depth of structure from base of rail to invert level
- Requirements for soil backfill and cover surrounding the structure
- Depth requirements for ballast above the top of the structure
- Alignment and skew angle, if it applies
- Subgrade width and embankment slopes
- Existing soil and foundation conditions
- A 10-foot vertical inside clearance

5.2 Pedestrian Underpasses Type Selection

A pedestrian underpass, also known as a box, is a structure with rectangular shaped openings through an embankment primarily for pedestrian or equipment access. Different types of underpasses that have been proposed to PCJPB include:

1) **Precast Concrete Box** – This type comprises of box sections with tongue and groove joints. The selection of the joint system used is important if the loads are to be distributed across the joint. Post-tensioning can be used between each joint section, over the box length, or a combination of both to resist differential settlement, to promote sealing, and to provide load distribution. Simple and fast installation of the precast concrete sections may be achieved with this type of underpass.

2) **Soldier Pile Walls with Precast Roof Slab** – Soldier pile walls consisting of augured/ soil-cement holes, typically with alternating H-piles as reinforcement, offer little impact to the site until final excavation and roof installation. This structure type resolves differential settlement issues. Multiple components to this installation require longer time to furnish. Soldier piles may also be substituted with secant piles with alternating concrete-filled reinforced and un-reinforced piles. Shoring and over excavation is minimized. Construction is suited for working around railroad tracks in operation.

3) **Cast-in-Place Concrete Box** – Casting the underpass in a series of concrete pours achieves a continuous monolithic structure. Water sealant may be automatically integrated in the mix design or with an outside waterproof membrane. However, extensive construction staging is required, particularly around railroad tracks, to provide ample curing time for the required concrete strength to develop.

4) **Contech Multi-Plate Arch with Interior Facing** – This arch can be assembled onsite by a Contech contractor and hoisted into place over a weekend closure of railroad operations. The metal sections are assembled nearby and can be lifted into place in a short amount of time. Although the material cost for the Contech arch is the least expensive of the alternatives, significant excavation, shoring, and extensive interior treatments are needed.

5) **Conspan Arch Pedestrian Underpass with Cast-in-Place Footing** – Cost savings may be achieved by minimizing the on-site labor required for cast-in-place box construction with this method. Footings are formed without special construction techniques and Conspan manufactured precast arches may be set in place quickly. Precast headwalls may also be manufactured to meet project needs.

Refer to Figure 5.1 for typical sections and Table 5.1 for advantages and disadvantages of each structure type. Some structure types may be preferred over others depending on site conditions, material and construction cost, interior aesthetic treatments, utility locations, railroad track operating closure periods, and public preference. Aside from the structure types listed above, PCJPB welcomes other possible alternatives that may be appropriate for a project.

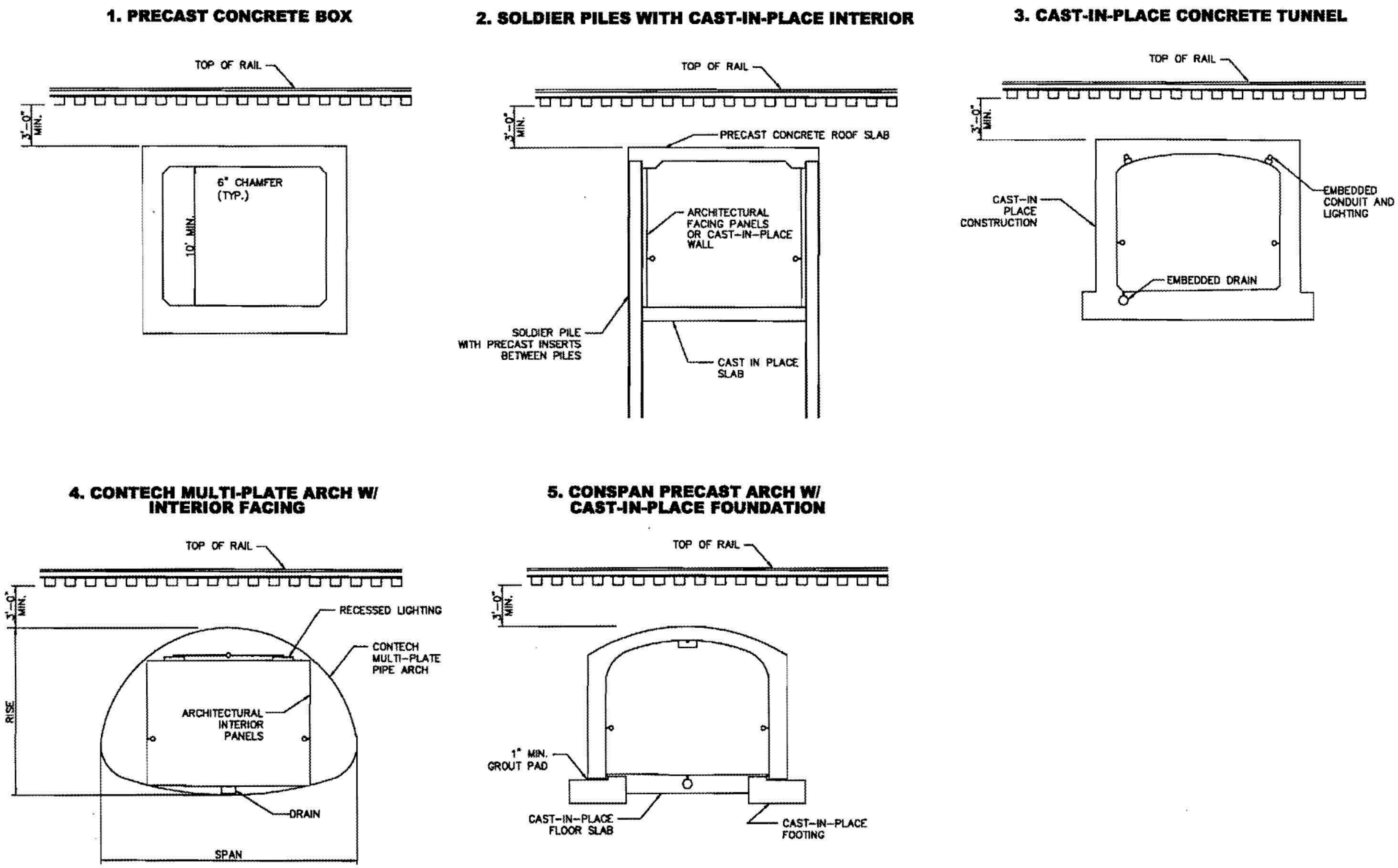


FIGURE 5.1 – PEDESTRIAN UNDERPASS TYPES

TABLE 5.1 – ADVANTAGES AND DISADVANTAGES OF BOX TYPES

ADVANTAGES	DISADVANTAGES
Type 1: Precast Concrete Box Tunnel	
1. Precast quality control 2. Possible completion over weekend closure of railroad 3. Possible phased construction 4. May be jacked under existing tracks	1. Critical excavation & haul needed 2. Large crane capacity for lifting heavy sections 3. Possible leaking & sealant problems between joints 4. Foundation & excavation need to be prepared quickly 5. Keyed sections require construction in one direction 6. May require shoring (separate from structure)
Type 2: Secant Pile Walls with Precast Roof Slab	
1. Possible completion over weekend closure of railroad if piles are placed beforehand. 2. No shoring required. Secant piles act as shoring 3. Less excavation required than other types 4. Minimal heavy lifting 5. Foundation built-in with pile placement 6. Resolves differential settlements 7. Can be used as temporary bridge support for track rather than a shoofly 8. Seals off water intrusion into the excavation	1. Waterproofing under high water table may be more difficult 2. Interior work necessary after weekend closure 3. Pile alignment needs to be accurate 4. Assembly of more components to install tunnel 5. Specialty sub-contractor required for installation
Type 3: Cast-in-Place Concrete Tunnel	
1. Watertight seal possible 2. Open excavation and shoring possible 3. No heavy lifting components required 4. Differential settlement not a critical	1. Forming and shoring required 2. Extensive staging & phased construction necessary 3. Crossovers required for switching tracks during construction or shoofly 4. More time required for construction due to concrete cure time
Type 4: Contech Multi-plate Arch with Interior Facing	
1. Light section may be lifted in one section 2. Possible completion over weekend closure of railroad 3. Compaction requirements may be reduced with cementatious flowable fill 4. Phased construction possible 5. Full length of preassembled tunnel shapes 6. Additional interior room for drainage and conduit/ Fiber optics placement 7. Waterproofing may be completed prior to installation	1. Rapid excavation and haul critical 2. Extensive finish work required on tunnel interior 3. Compaction in 8-12" lifts may be required 4. Increased depth of excavation and/or cover required to develop Cooper E80 train loading strength in arch 5. Buoyancy control may be difficult
Type 5: Cast-in-place Footing with Precast Arch Wall & Roof	
1. Precast quality control of arch 2. Interior may be aesthetically finished by precaster 3. Simple footing forms and reinforcement 4. Lighter sections to be lifted 5. Less on-site labor for reinforcing and forming	1. Excavation and haul critical 2. May not be possible to complete over weekend closure. 3. Cast-in-place footing requires extended cure time 4. Waterproofing more difficult at arch joints 5. Shoring required

5.3 Structural Design of Pedestrian Underpasses

The structure design of pedestrian underpasses shall follow the design requirements of AREMA tor the type of materials being considered and the loading provisions for the material selected.

Provisions under the bulleted items of this Chapter should be used as a guideline, in the development of a cast-in-place pedestrian box for conceptual design. A more rigorous analysis and design procedure should be employed for final design.

- Equations in Figure 5.2 may only be used for preliminary and conceptual studies to calculate the loading conditions acting on the box. The pedestrian underpass shall be analyzed as having rigid joints between slabs and walls for a cast-in-place box type of pedestrian underpass, and with the positive and negative bending moments determined by elastic theory.

5.3.1 Loading

(a) Dead Load

Dead loads to be considered for the box design consist of the weight of the track, ballast, and fill on the top slab of the structure. The dead load shall be uniformly distributed to the top surface of the pedestrian underpass.

Lateral earth pressure coefficients shall be recommended by the geotechnical engineer specific to the site to be used in the analysis.

- For preliminary design when data is not available, minimum and maximum earth pressure coefficients (acting laterally) can be assumed to be 0.33 and 1.0, respectively, and checked for both conditions when the equations in Figure 5.2 are used. Lateral pressures shall be distributed uniformly over the height of the pedestrian underpass, equal and opposite in direction.

(b) Live Load

Design live load shall be Cooper E-80.

The lateral live load surcharge for final design shall be developed per recommendations from AREMA by lateral surcharge pressures.

Long-term compaction by traffic may be considered over time, with varying soil moisture content below the pedestrian underpass. Design loading shall be based on a range of soil mass density, earth pressure coefficients and hydrostatic conditions.

For pedestrian underpasses that are designed for track live load, the distributed surcharge of one track shall be considered the same over the full length of the structure within the PCJPB Right-of-Way. The live load shall be treated in the same manner.

- For preliminary design and the use of equations in Figure 5.2, the live load on a pedestrian box shall be distributed accordingly for train loads that are perpendicular to the pedestrian box. No increase in load shall be used for multiple track loadings—

lateral load distribution for one track shall be limited to the track centers of multiple tracks.

- For preliminary design, lateral pressure due to railroad surcharge shall be computed per Figures 5.2.

(c) Impact Load

For final design, the impact loads shall be distributed on the top slab of the pedestrian box, similar to the live load distribution. The sides of the box shall not be affected by impact loading.

Impact loading shall range from 40% of the live load for a cover height of 18 inches, to 0% of the live load for a cover height of 10 feet.

For preliminary design and the use of equations in Figure 5.2, the live load impact shall be applied as described for final design above.

(d) Seismic Loads

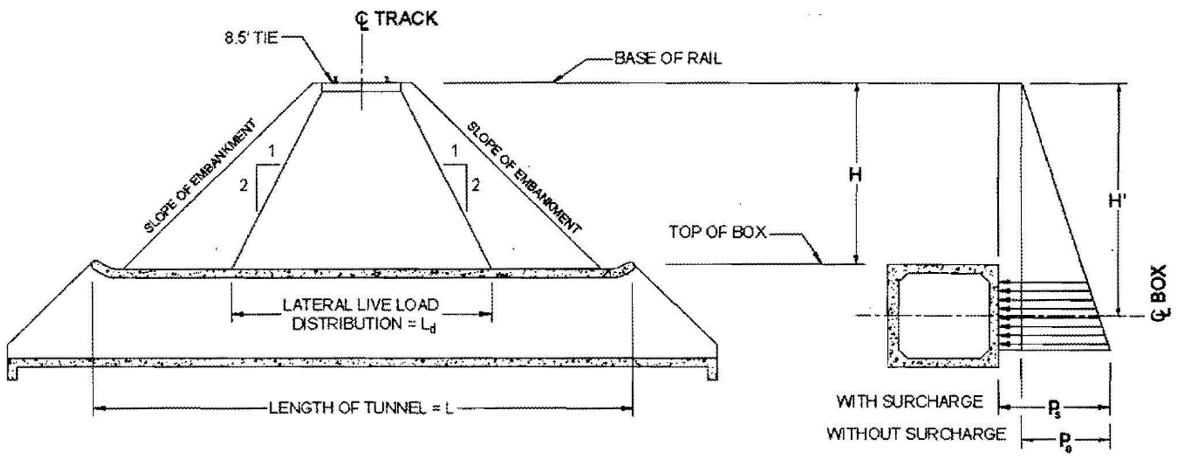
Earthquake loading shall be considered for pedestrian box design. However, boxes by nature of their function are presumed to be of a design resistant to seismic loads. Any pedestrian underpass regardless of material, including arched shaped or of proprietary designs shall be designed for seismic loading.

The magnitude of the seismic load shall be calculated as a function of the vertical acceleration component of the seismic event. This load is to be applied as a uniform dead load at the centroid of a fill section containing the pedestrian underpass. Risk factors are to be used to determine acceptable values for allowable stresses.

Refer to Chapter 4 of these Standards for specific seismic requirements pertaining to railroad structures.

(e) Hydrostatic Pressure

Due consideration shall be made to high water table situations. The pedestrian underpass shall be designed to resist the effects of buoyancy. A factor of safety of 1.5 shall be applied in buoyancy calculations.



- **Live Load to be Considered:** E-80
- **Loads on Top Slab:**

$$W = W_{LL}\left(1+\frac{I}{100}\right)+W_{DL} = \text{ Uniform Load in psf}$$

$$W_{LL} = \frac{80000\ lbs}{5\times L_d} = \text{ Uniform Load in psf}$$

$$W_{DL} = W_e H + \frac{200}{L_d} + W_s = \text{ Uniform Load in psf}$$

$$I = \text{ From } 40\% \text{ at } H = 18 \text{ inches}$$
$$\text{To } 0\% \text{ at } H = 10 \text{ feet}$$

- **Load on Walls:**

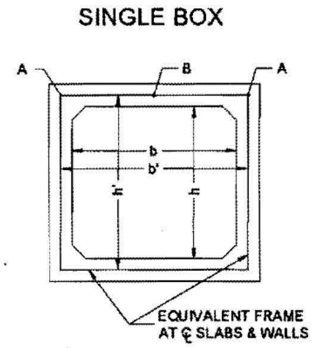
$$P_e = k_e W_e H' = \text{ Uniform Load in psf}$$

$$P_s = k_s W_e\left(H' + \frac{80000}{5 W_e L_d}\right) = \text{ Uniform Load in psf}$$

$$k_e = 0.33 \text{ min., } 1.0 \text{ max.} \qquad k_s = 0.33$$

FIGURE 5.2 – DESIGN OF PEDESTRIAN UNDERPASSES

- **Design Equations for Single Box (per Foot of Pedestrian Underpass Length)**



$$Max\quad M_B = \frac{Wb^2}{24}\left(\frac{1+3k}{1+k}\right) - \frac{P_e h^2}{12}\left(\frac{k}{1+k}\right) = in\quad lb-ft,\ use\quad min.\ value\ of\quad P_e$$

$$Max\quad M_A = \frac{Wb^2}{12}\left(\frac{1}{1+k}\right) + \frac{P_s h^2}{12}\left(\frac{k}{1+k}\right) = in\quad lb-ft,\ use\quad max.\ value\ of\quad P_e\quad or\quad P_s$$

$$V_A = \frac{Wb}{2} = in\quad lbs$$

- **Design Equations for Double Box (per Foot of Pedestrian Underpass Length)**

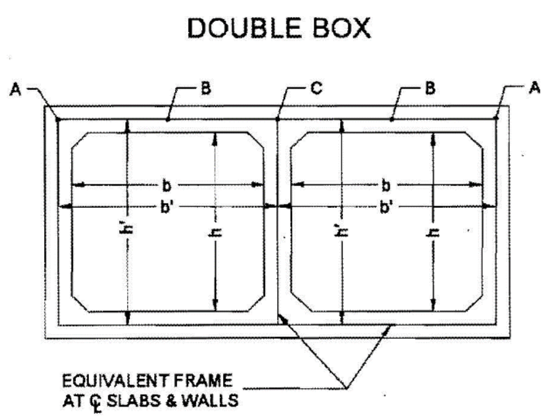


FIGURE 5.2 (CONTINUED) – DESIGN OF PEDESTRIAN UNDERPASSES

$$Max\ \ M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right)+\frac{P_s h^2}{6}\left(\frac{k}{1+2k}\right) = in\ \ lb-ft,\ \ use\ \ max.\ \ value\ \ of\ \ P_e\ \ or\ \ P_s$$

$$Max\ \ M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right)+\frac{P_e h^2}{6}\left(\frac{k}{1+2k}\right) = in\ \ lb-ft,\ \ use\ \ min.\ \ value\ \ of\ \ P_e$$

$$Max\ \ M_C = \frac{Wb^2}{12}\left(\frac{1+3k}{1+2k}\right)-\frac{P_e h^2}{12}\left(\frac{k}{1+2k}\right) = in\ \ lb-ft,\ \ use\ \ min.\ \ value\ \ of\ \ P_e$$

$$V_A = \frac{Wb}{4}\left(\frac{2+3k}{1+2k}\right)+\frac{P_s h^2}{4b}\left(\frac{k}{1+2k}\right) = in\ \ lbs.,\ \ use\ \ max.\ \ value\ \ of\ \ P_e\ \ or\ \ P_s$$

$$V_C = \frac{Wb}{4}\left(\frac{2+5k}{1+2k}\right)-\frac{P_e h^2}{4b}\left(\frac{k}{1+2k}\right) = in\ \ lbs.,\ \ use\ \ min.\ \ value\ \ of\ \ P_e$$

Notations:

b	= Width of box opening
b'	= Horizontal distance between center lines of box walls
h	= Height of a box opening
h'	= Vertical distance between the centerlines of box top and bottom slabs
H	= Vertical distance from base of rail to top of box
H'	= Vertical distance from base of rail to center of box opening
I	= Impact load applied on top of box, as a percentage of W_{LL}
I_s	= Moment of inertia of the top slab gross section, per foot of tunnel length
I_w	= Moment of inertia of the wall gross section, per foot of tunnel length
K	= Ratio of S to R
k_e	= Coefficient of active earth pressure for embankment fill without surcharge loading
k_s	= Coefficient of active earth pressure for embankment fill including surcharge loading
L_d	= Lateral live load distribution length
M_A	= Maximum negative moment at the exterior corner of the box, per foot of tunnel length
M_B	= Maximum positive moment at the center of the top slab, per foot of tunnel length
M_C	= Maximum negative moment in the top slab at the top of the center wall, per foot of tunnel length
P_e	= Uniformly distributed lateral earth design load acting on the sides of the box
P_s	= Uniformly distributed lateral earth + surcharge load acting on the sides of the box
R	= Ratio of b' to h'
S	= Ratio of I_s to I_w
V_A	= Maximum vertical shear in top slab, at the face of support near an exterior corner per foot of culvert length
V_C	= Maximum vertical shear in top slab, at the face of support near a center wall per foot of tunnel length
W	= Total uniformly distributed load on top of box, combination of W_{LL}, W_{DL}, and I
W_{LL}	= Uniformly distributed live load on top of box
W_{DL}	= Uniformly distributed dead load on top of box
W_e	= Mass density of embankment fill (120 lbs/ft^3)
W_s	= Mass of concrete per square foot of top slab area

FIGURE 5.2 (CONTINUED) – DESIGN OF PEDESTRIAN UNDERPASSES

(f) Combination of Loads and Loading Cases

All elements of the pedestrian underpass shall be designed for a combination of lateral soil, groundwater, and surcharge loads acting in conjunction with vertical dead and live loads. Generally a box type structure is considered a rigid frame and will be designed for the vertical and horizontal earth load together with the combination of vertical live load, horizontal live load, and uplift pressure which give the greatest stresses in the various parts of the structure. See Figure 5.3. Structure shall be checked for construction loading conditions.

Upon request by PCJPB, the underpass shall be designed to accommodate future expansions (i.e., increased horizontal and vertical clearances, special equipment loading besides pedestrians and for the addition of future tracks).

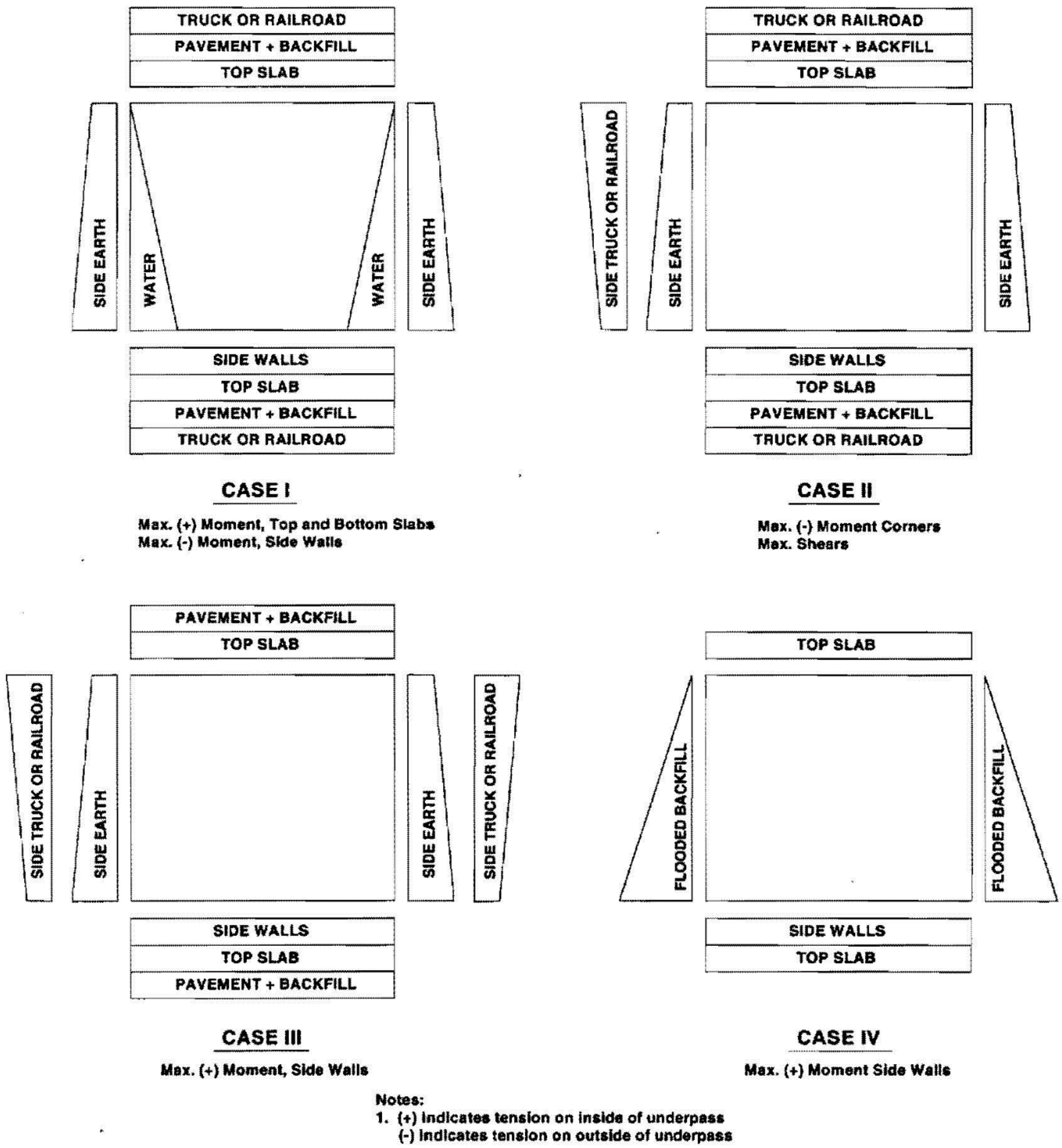


FIGURE 5.3 – LOADING CONDITIONS FOR DESIGN OF PEDESTRIAN UNDERPASS

5.3.2 Design Requirements

The same underpass cross-section shall be used throughout the box whenever possible. Consideration shall be given to provide for future tracks.

- For preliminary design the engineer shall design for a minimum thickness of 12 inches for top and bottom slabs, and walls depending on loading conditions and height of cover.

Cast-in-place construction may require larger wall and slab thickness to ensure proper concrete placement.

(a) Joints

Placement of joints must be carefully considered to minimize vertical and longitudinal movements, especially for long pedestrian underpasses or in sites with high fill material.

All joints shall have water tight joints and be waterproofed.

Joints shall be spaced away from the centerline of track. Due consideration shall be given to assure load transfer across joints.

(b) Reinforcement

The Engineer shall provide sufficient transverse reinforcement to satisfy loading requirements. Minimum longitudinal reinforcement in top slabs, bottom slabs, and walls shall be:

Fill depths ≤ 10 feet	0.4% of concrete cross sectional area
10 feet < fill depths ≤ 100 feet	Increase proportionally to 1% of concrete cross sectional area

(c) Foundation

Pedestrian boxes shall be placed on well-graded and level foundation surfaces. For precast underpasses, the foundation shall include a minimum of 12-inch-deep layer of compacted crushed stone. Where finer compacted material is needed, a sand layer may be used on top of the crushed stone bed. Cast-in-place construction may not need the crushed stone bed foundation. The designer shall provide recommendations for appropriate foundation material. A mud slab may be required under the bottom slab of a box to facilitate construction if ground water and a muddy base are encountered.

5.4 Special Provisions

5.4.1 Waterproofing

Waterproofing around a pedestrian underpass shall prevent water seepage into concrete joints and the interior of the underpass if there is the possibility of a high water table. Waterproofing is required for all PCJPB pedestrian underpasses that may come in contact with any ground water. Based on the maximum groundwater level anticipated, the designer shall recommend the most appropriate sealant method. Refer to AREMA, Chapter 29 for waterproofing requirements and types.

Selection of waterproofing type shall be based on its resistance to oil, diesel agents, and other contaminated materials in the soil if they are present at the site of the box.

In most cases new concrete shall be cured for a minimum of 7 days before applying waterproofing system. Refer to the manufacturer’s recommendations for accurate installation of waterproofing.

Different types of waterproofing have been proven to work effectively in railroad environments, such as roll-on waterproofing material or membrane types.

(a) Types of Waterproofing

i. **Slurry Coat Crystallization:** A layer of slurry coat may be applied to joint surfaces between pours of box elements. Dry powder compounds are applied to concrete in the slurry coat, forming fibrous crystals throughout the pours. The 2.0 lb/sq. yd. slurry coat substance will not deteriorate under normal conditions and not subjected to effects from humidity, ultraviolet radiation, and oxidization. Typical temperatures suitable for the slurry coat range from -32°C to 130°C.

The slurry coat does not require dry weather to crystallize, surface priming or leveling, and may be applied to either negative or positive pressure sides. The crystallization method is applicable to existing concrete structures and precast elements. Manufacturers for slurry coats include XYPEX, VANDEX, SPECON, and others. Designers shall verify vendors and quotes for specific project site.

ii. **Waterproofing Membrane:** The most effective method to protect a pedestrian underpass is by providing a waterproofing membrane. Membrane material may be composed of (See Table 5.2):

- bitumen-treated cotton fabric or felt
- butyl rubber or EPDM
- rubberized asphalt with plastic film
- cold liquid-applied elastomeric membrane with a primer.

When the exterior of the underpass has a waterproofing membrane, a protection course shall be applied prior to the placing of backfill. Backfill shall be selected and placed carefully to prevent damage to the waterproofing system. Flexibility and expansion shall be provided in waterproofing membrane at pedestrian underpass joints where deflection may cause stretching in the material.

The designer shall advise PCJPB of potential sealing problems on projects when necessary. Refer to Section 5.2 for using appropriate waterproofing for different structure types. Every effort shall be made to match existing waterproofing system types when widening or retrofitting an existing PCJPB pedestrian underpass.

TABLE 5.2 – WATERPROOFING MEMBRANE TYPES

Type	Temperature of Application	Properties	Application Procedure
Bituminous Membrane	> 50° F (10° C)	• Coal-tar pitch cannot be heated above 300° F • Asphalt cannot be heated above 350° F	1. Mop the surface at ½ gallon per 100 sq. ft. of surface. 2. Layers of felt/fabric will be lapped on with a min. of 12-inch splice. *See AREMA, Ch. 29.*

			3. Apply anti-bonding paper and galvanized metal over expansion joints. 4. Patching only by permission of Engineer.
Butyl Rubber/EPDM	> 10° F (-12° C)	• Requires a strip area of 36 inches long. • Membrane thicknesses =0.06 inch, 0.09 inch, or 0.12 inch • Hardness =60 ± 10 durometer • Tensile strength = 1,300 psi • Puncture resistance = 70 lbs. *Refer to AREMA, Chapter 29 for other properties.*	1. Apply adhesive and roll membrane tightly over surface. 2. Clean all seams, laps, and splices. See *AREMA, Ch. 29*. 3. Apply anti-bonding paper and galvanized metal over expansion joints. 4. Patch over punctures with minimum 4-inch overlap.
Rubberized Asphalt with Plastic Film	> 50° F (10° C)	• Membrane thickness = 0.06” • Max. Permeability = 0.1 perms • Min. Puncture Resistance = 40 lbs. • Peel Adhesion = 5.0 lb/inch of width after 7 days dry • Crack bridging = 100 cycles 0.25”	1. Clean and dry surfaces. 2. Apply primer at a rate of 100-250 sq. ft. per gallon. Allow primer to dry for an hour. 3. Overlap rubber asphalt with plastic film at 2½ inches, and with preformed board at 6 inches. 4. Apply anti-bonding paper and galvanized metal over expansion joints. 5. Trowel the perimeter of the membrane with cold applied asphalt mastic.
Cold Liquid-Applied Elastomeric Membrane	Between 32 – 104° F (0 – 40° C)	• Membrane Thickness = 100 mils • Adhesion/pull off values = 100 psi for concrete, 290 psi for steel • Min. Tensile Strength = 930 psi • Crack bridging = pass at 25 cycles 0.0625”	1. Dry all surfaces. 2. Spray membrane fully over all the sides. Brush on touch-up as needed. 3. Cure fully before covering the surface. 4. Put on protective cover (either Portland cement concrete or asphalt plank/ panels) to prevent damage from the ballast.

5.4.2 Drainage

The surface of the top slab in contact with the backfill may be sloped towards the sidewalls for drainage. Interior walkway slabs shall be crowned to slope towards the sidewalls and then drained longitudinally and be collected into a drain system.

Pipe drains may be required to be designed in the backfill adjacent to the sidewalls of the pedestrian underpass. Horizontal drainpipes shall be perforated and at least 8 inches in diameter.

Pedestrian underpasses that are located below the adjacent grade shall be provided with a positive method of eliminating water that may seep into the underpass (i.e., pumping system). Discharge of drainage water shall be collected at an approved discharge point.

5.5 Construction of Pedestrian Underpasses

Staging and construction scheduling shall be carefully planned to minimize impact on PCJPB rail service.

The PCJPB's construction windows for projects that will interfere with rail operations shall be limited to weekends (from Friday evening 9 PM through Monday morning 3 AM) and night time work. During this window, active tracks will be non-operational and removed from service. Tracks, ballast, ties and all service-related components shall be replaced for operation prior to the end of the allotted construction window.

When staging is required for construction outside the specified window, special arrangements must be made with PCJPB. Staging may be required particularly for cast-in- place structures to achieve curing strength. One of three staging methods shall be implemented if a project entails weekday construction hours: (1) construction of the structure 15 feet outside of active railroad traffic, (2) phase construction so one track remains active, or (3) construct a temporary shoofly or bridge to accommodate railroad traffic.

The engineer shall approve all backfill and bedding materials used for underpasses. All backfill and bedding material shall be free from brush and other organic materials. Wet or impervious materials are not to be used for backfill material of PCJPB structures, unless otherwise approved by the engineer.

CHAPTER 6

RETAINING WALLS

CHAPTER 6: RETAINING WALLS

Purpose

This guideline was created to provide standards and requirements regarding design and construction of new or modified permanent retaining walls that affect the tracks and property of the Peninsula Corridor Joint Powers Board (PCJPB). Design engineers can use these guidelines as a basis for design. Abutments are also discussed in this Chapter since soil loading conditions are similar to that of retaining walls. For temporary shoring walls, refer to the PCJPB Guideline for Excavation Support Systems.

6.1 General Requirements

Design of retaining walls shall comply with the appropriate parts of the current AREMA Manual of Railway Engineering. Applicable sections of the manual include Chapter 8, Part 2, 5, 6, 7, 20, and 22, among others.

The addition of future tracks along the PCJPB Right-of-Way shall be explored prior to the design of retaining walls along the tracks. Provisions must be made for at least four tracks along the PCJPB Right-of-Way.

The designer shall base the retaining wall design on soil conditions from available reports or a new geotechnical report shall be generated if warranted. A log of test borings shall accompany the foundation report and the drawing plan set shall include the log of borings.

6.1.1 Design Considerations

The selection and design of retaining walls shall be based on:

- The purpose of the structure
- Stresses and loads to be applied to the wall
- Depth of structure from base of rail to invert level
- Requirements for soil backfill and compaction
- Existing soil and foundation conditions
- Required height of the wall

6.2 Retaining Walls Type Selection

A retaining wall provides lateral support for earth fill, often times supporting live loads adjacent to the soil mass. The different types of retaining walls that may be used along the Caltrain railroad include:

1) **Gravity Wall –** The gravity wall relies on its weight to for stability. For a concrete gravity wall only temperature steel is required for reinforcement in gravity walls.

2) **Semi-Gravity Wall –** This type is an intermediate between a gravity wall and a cantilever wall. For a concrete semi-gravity wall some steel reinforcement is needed along the back and lower side of the toe.

3) **Cantilever Wall –** This type wall utilizes cantilever action to retain the mass behind the wall from assuming a natural slope. A concrete wall of this type will require steel reinforcement is needed for this L-shaped configuration.

4) **Counterfort Wall –** This type of wall consists of vertically reinforced counterforts laterally supporting a reinforced vertical face slab buried in backfill, which in turn, all supported by a reinforced base slab.

5) **Buttress Wall –** Buttresses support a vertical face slab, as in counterfort walls, but exposed on the face of the wall rather than buried in the backfill.

6) **Crib Wall –** Relying on the weight and strength of earth fill, crib walls are comprised of an earth-filled assembly of separate structural units.

7) **Mechanically Stabilized Earth (MSE) Wall –** More economical, MSE walls are formed by reinforcing the backfill with reinforcement straps.

The designer shall consider site conditions, load demands including seismic, and cost efficiency when selecting the appropriate type of retaining wall to be used. PCJPB prefers designs that are most compatible with railroad operations and that can be constructed with a minimum amount of disturbance to train traffic.

The PCJPB Right-of-Way shall be verified by professional land surveyors, prior to construction of any structure, especially if the wall is located on the property line. Retaining walls along railroad Right-of-Way may require temporary construction easements on adjacent property to facilitate construction. Permitting and easements shall be obtained through the Real Estate Department at PCJPB. The designer shall make every effort to eliminate the acquisition of temporary and permanent easements.

All earth retention structures owned by Caltrain shall be designed in accordance with criteria described in Chapter 8, Part 5 of the most current AREMA Manual, and any special provisions made by PCJPB.

6.2.1 Factors for Selecting Retaining Wall Types

Include all appropriate earth retaining systems in the concept design development documents that will promote a selection of a retaining wall system that satisfies the requirements of the PCJPB and the site conditions. Earth retaining systems that lead to cost savings may be most favored. PCJPB will make final decisions on the type of system to be used based on the designer's recommendations. The following items will be considered in the selection of the retaining wall structure:

- **Cost –** PCJPB generally prefers the most cost-effective alternative for retaining wall structures, yet still meeting height and length design requirements. For construction costs, mechanically stabilized earth walls (MSE) may be less expensive than cast-in-place gravity walls, especially in locations that require additional fill material. Generally, gravity walls are the most basic and common alternative but can become expensive when supporting railroad surcharge loadings (depending on height).

- **Location –** The location of retaining walls will affect operations and constructability of certain wall types. Walls shall be of a type that will minimize impact to PCJPB operations.

When sufficient construction space or access is not permitted, select a wall type that requires the least Right-of-Way acquisition. In most cases, cast-in-place type wall installation may require an additional encroachment off PCJPB property if it is located on a Right-of-Way line. The designer shall inform PCJPB of potential construction easements or permitting requirements necessary for the construction of a retaining wall.

- **Aesthetic Conditions –** Retaining walls for retrofit or widening projects shall match existing aesthetic conditions whenever possible. New structures will conform to the requirements of PCJPB and local City or County agencies. Wall types and aesthetic treatments shall be uniform throughout a project (i.e., cast-in- place walls to resemble the finishing of adjacent, existing MSE wall when visible to the public). Side slopes of the wall shall be graded to provide a pleasing profile.

- **Maintenance –** Costs incurred for maintenance shall be considered when selecting the type of retaining wall structure. Wall types selected shall require minimal maintenance and replacements. For instance, graffiti-free coating spray may be used to minimize graffiti vandalism during the service life of a wall.

- **Constructability –** The construction of walls along the PCJPB corridor are typically constrained by the active operating track and the available Right-of- Way available for construction. Consideration shall be given to the operations of the PCJPB and to minimize any interference with train operations.

6.3 Structural Design Requirements

6.3.1 Wall Design Heights

Retaining wall heights will be designed to provide for future raises in the track. The height for retaining walls shall vary linearly from top of rail (T/R), in relation to the distance away from the centerline (CL) of track (as the distance away from the centerline of track increases, the required height of the retaining wall can decrease). At a 10 feet minimum distance from the centerline of track, the retaining structure shall be at the same elevation as top of rail. The wall height requirement may be reduced to 8 inches below top of rail, at the maximum distance of 13 feet away from the centerline of track. Refer to PCJPB Track Standard TS-1025 and TS-1030 for cross section of track to be used to establish wall heights parallel to the track, greater than 13 feet from centerline of track.

6.3.2 Loads

Pressure loading on the retaining wall is dependent upon the type of structure, the type of fill, and the location of the applied live load.

(a) Live Load

Live loads that may affect the design of earth retention structures consist of trains, street or highway traffic, or other loading combinations. Live loads due to railroad trains shall be determined per PCJPB's requirements and in most cases Cooper E-80 axle loading shall be used as found in Chapters 8 and 15. The application of the railroad surcharge loading shall be per AREMA Chapter 8, Section 5.3. Highway-traffic live loading shall be applied in accordance with Caltrans Design criteria. Construction loads shall be considered to account for future maintenance operations of the PCJPB.

Other live loads, including those due to structures, shall be analyzed on an individual-case basis and applied either as a uniform surcharge, point load, or line load.

(b) Applied Loading Exclusive of Earth Pressure

Chapter 8, Section 5.3.1 of the AREMA Manual describes other load applications (i.e., superimposed dead loads and live load surcharge) besides earth pressure.

- **Perpendicular to the track (such as an abutment) –** The track loading is uniformly distributed on the ballast over the width of the tie when structures or abutments lie almost perpendicular to the track center. This surcharge loading distribution increases with depth on a 1 horizontal to 2 vertical slope on both sides with surcharge from adjacent tracks not being permitted to overlap.

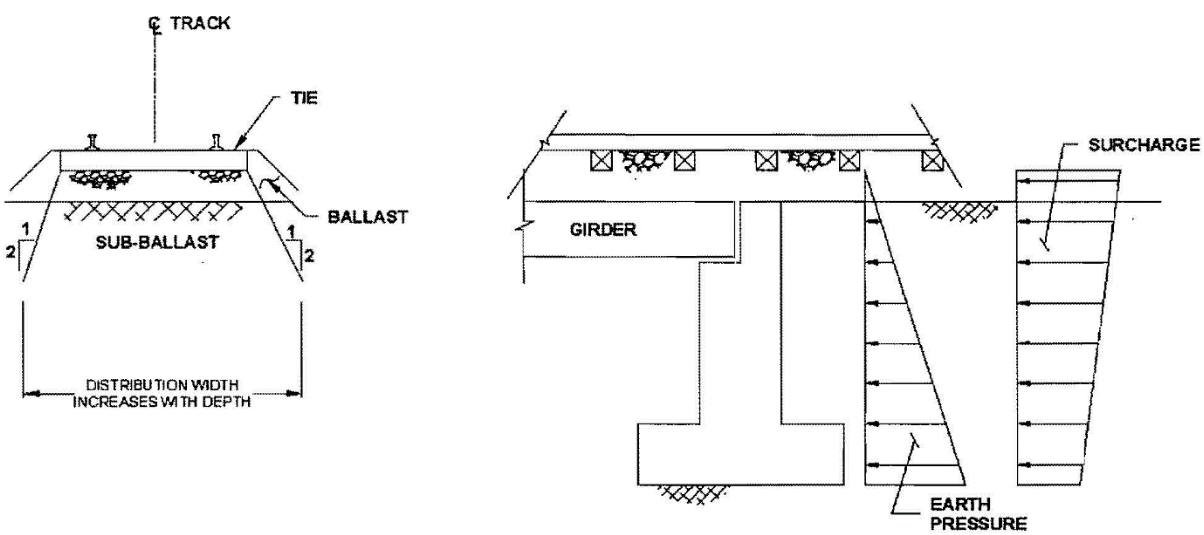


FIGURE 6.1 – TRACK PERPENDICULAR TO SUPPORTING STRUCTURE

- **Parallel to the track –** When a wall is parallel to the tracks, the surcharge load is distributed uniformly over the width of the tie. Pressures on the retaining wall, Ps caused by a continuous strip of surcharge load q (pounds per square foot) can be analyzed by:

$$P_s = \frac{2q}{\pi}\left(\beta + \sin\beta \sin^2\alpha - \sin\beta \cos^2\alpha\right)$$

where α and β are defined in Figure 9.2.

The Boussinesq pressure distribution for a strip load may be used for all surface surcharge loads. The pressure distribution may be simplified into a rectangular distribution with a magnitude 80% of the maximum Boussinesq pressure or other some other method upon approval of the PCJPB.

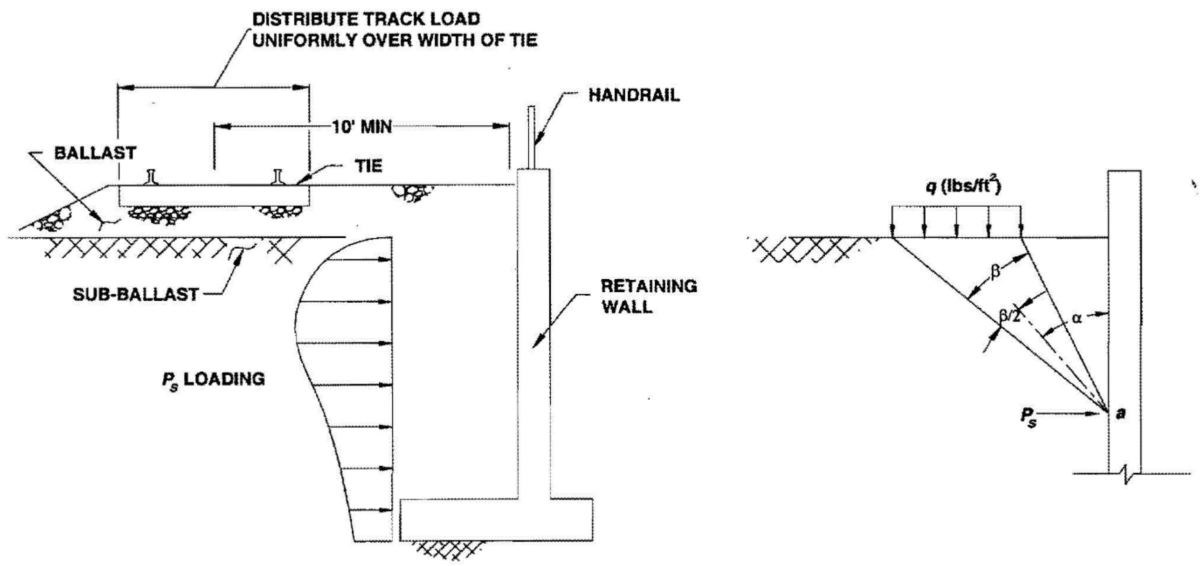
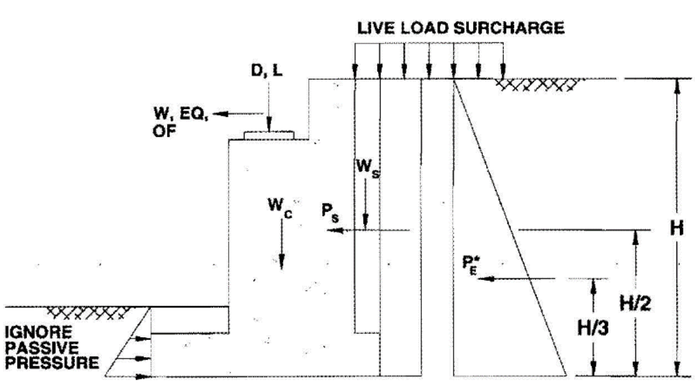


FIGURE 6.2 – TRACK PARALLEL TO SUPPORTING STRUCTURE

(c) Loading Combinations

Various loads and forces shall be combined in accordance with the most current AREMA Manual. Factored loading shall be used when using the Load Factor Design (LFD), and unfactored loading when using Allowable Stress Design (ASD).

(d) Forces Acting on Typical Abutment



D = Dead Load from superstructure
L = Live Load from superstructure
W = Wind Load on structure
EQ = Earthquake (Seismic) Load on structure
OF = Other Forces (including rib shortening, shrinkage, temperature and/or settlement of supports (assume 10% of Dead Load)
H = Height of abutment
W_C = Weight of concrete abutment
W_S = Weight of soil behind abutment
P_S = Lateral Soil Pressure from live load surcharge
P_E = Lateral Pressure due to Earth Embankment

*Active Earth Pressure without earthquake is usually taken as H/3. Dynamic Pressure due to earthquake is assumed to be 0.60H from the base. Combined effect is assumed by most engineers to be 0.5H.

FIGURE 6.3 – FORCES ACTING ON ABUTMENT

(e) Forces Acting on Typical Abutment

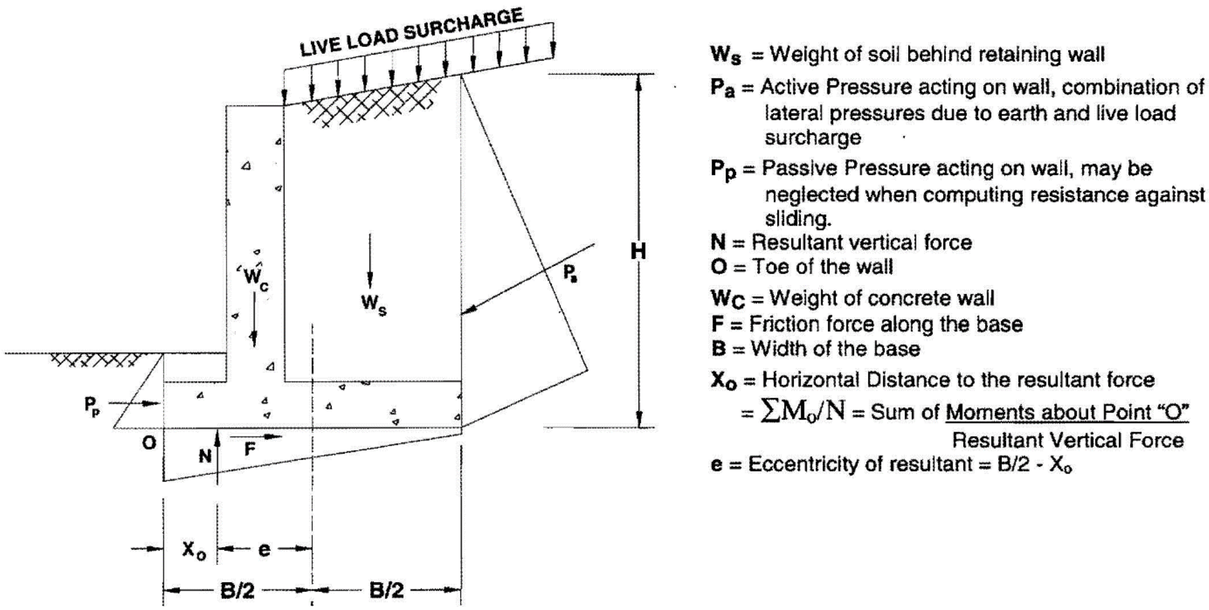


FIGURE 6.4 – FORCES ACTING ON RETAINING WALL

(f) Backfill Pressure

Backfill may be considered undisturbed ground or fill material behind a retaining structure. Classification of backfill types and their properties that may be found on PCJPB structures are shown in Table 9.1. Types 4 and 5 backfill are generally not allowed by PCJPB, and their use requires approval.

TABLE 6.1 – BACKFILL TYPES AND PROPERTIES FOR RETAINING WALLS

Type	Backfill Description	Unit Dry Weight (lb/ft^3)	lb/SF	Cohesion "C" Angle of Internal Friction
1	Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel or broken stone).	105	0	33° 42' (38° for broken tone)
2	Coarse-grained soil of low permeability due to admixture of particles of silt size.	110	0	30°
3	Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones.	125	0	28°
4	Soft or very soft clay, organic silt; or soft silty clay.	100	0	0

5	Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.	120	240	0

Appropriate structural backfill and compaction will be selected based on the design of the engineer. If after conducting geotechnical investigation and existing backfill is unsuitable for abutments or retaining structures, backfill shall be replaced with material appropriate for the structure. Higher quality backfill material that reduces active soil pressure and lateral loading on the structure may be preferred over an elaborate structural solution.

Other provisions and calculations for backfill pressure shall be computed using formulas found in AREMA, Chapter 8, Section 5.3. The Rankine-Coulomb Theory may be used under the following conditions: when backfill is cohesionless, surface of backfill is flat, and either without surcharge or uniformly distributed surcharge loading.

(g) Dynamic Earth Pressure

Dynamic earth pressure due to earthquake shall be considered using the Mononobe-Okabe Theory. The Mononobe-Okabe static analysis takes into account horizontal and vertical inertia forces acting on the soil, with the following assumptions:

1) The abutment/retaining wall is free to move sufficiently to mobilize the soil strength. If the abutment is rigidly fixed, the Mononobe-Okabe analysis produces very conservative values for soil forces.

2) The backfill is cohesionless with a friction angle Φ.

3) The backfill is unsaturated to prevent liquefaction.

The active force that results from the Mononobe-Okabe theory is given by:

$$P_{AE} = 0.5\gamma H^2 (1 - k_v) K_{AE}$$

where: γ = unit weight of soil
H = height of abutment/retaining wall
k_v = seismic acceleration coefficient in the vertical direction
K_{AE} = dynamic active earth pressure coefficient = $K_A + 0.75k_h$
K_A = static active earth pressure coefficient
k_h = seismic acceleration coefficient in the horizontal direction

Abutments must be designed for seismic loading, whereas other earth retaining structures need not be designed for earthquake loads if they do not support the PCJPB tracks and unless they have the potential to cause damage to essential facilities or other adjacent structures.

6.3.3 Retaining Wall Stability

Stability calculations are required for retaining structures per AREMA Manual, Chapter 8, Section 5.4.1. Earth retaining structures should satisfy all three stability criteria: overturning, sliding, and bearing pressure.

Overturning

Stability against overturning need not be checked if the resultant on the base is located within (1) the middle third of structures situated on soil, and (2) the middle half for structures situated on rock, masonry, or piles. Per AASHTO, overturning factors of safety of 2.0 or 1.5 are required for footings found on soil or rock, respectively.

Sliding

The factor of safety against sliding shall be at least 1.5. Friction resistance between the soil and the wall is calculated as the product of the normal pressure and coefficient of friction. For different subsoils, friction coefficient values may be taken as:

- coarse-grained soil without silt 0.55
- coarse-grained soil with silt 0.45
- silt 0.35

Bearing Pressure

Depending on soil strength and long-term settlement, bearing pressure can vary over the base of retaining structures. A factor of safety of 3 is generally used to achieve stability of the base against bearing capacity failure. Possible bearing pressure distributions acting on the footing, shown on Figure 9.5, shall be used to determine the maximum bearing pressure, q_{max}, while satisfying the following criteria:

$$\frac{R_1 q_{ult}}{FS} \geq q_{\max} \qquad (ASD)$$

$$\phi R_1 q_{ult} \geq q_{\max} \qquad (LFD)$$

where: q_{ult} = ultimate bearing capacity
R_1 = reduction factor due to inclined loads
FS = Factor of Safety
Φ = performance (strength reduction) factor
q_{max} = maximum bearing pressure due to unfactored loads (ASD)
= maximum bearing pressure due to factored loads (LRFD)

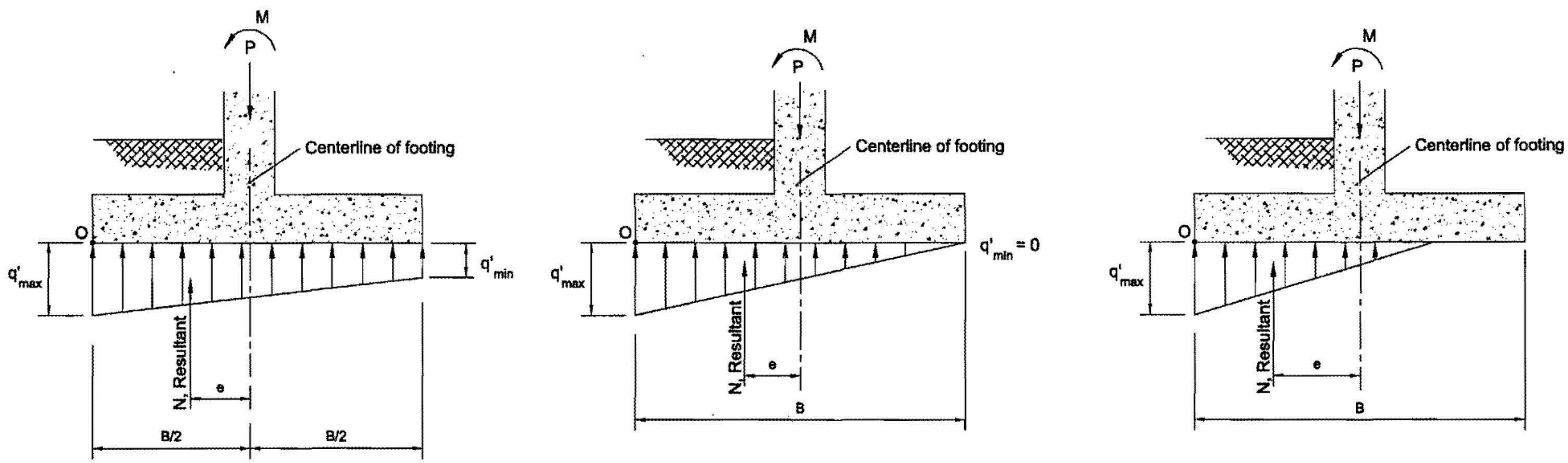
These bearing pressure distributions may be caused by permanent or temporary loads, such as wind or seismic, creating an uneven bearing pressure on the footing. The resultant bearing pressure, N, is a resultant of the loading conditions related to the eccentricity, *e*. For moment loads on square or rectangular footings, *e* = M/P, where M is the applied moment and P is the applied normal load on the footing.

If $e \leq B/6$, the resultant force acts within the middle one third-point and the eccentricity or applied moment is in the plane of the *B* dimension only, the minimum and maximum net bearing pressures, q'_{max} and q'_{min}, on a square, circular, or rectangular footings are given in Figure 9.5(a), Trapezoidal Distribution.

When $e = B/6$, the resultant force acts at the third-point of the footing and creates a triangular bearing pressure distribution as shown in Figure 9.5(b), Full Triangular Distribution (q'_{min} equals to 0 in the triangular distribution). Both the trapezoidal and triangular distributions are normally accepted in design because compressive stresses are present along the entire base of the footing.

However, if *e* > *B/6*, the resultant force at the base is outside the middle third- point. The pressure distribution is shown in Figure 9.5(c), Triangular Distribution. No tension is allowed between the footing and the soil thus resulting in no uplift. Large settlements at the toe and excessive tilting of the footing may result from this loading condition. This type of retaining wall design is not desirable in practice and shall preferably not be used.

(A) TRAPEZOIDAL DISTRIBUTION **(B) FULL TRIANGULAR DISTRIBUTION** **(C) TRIANGULAR DISTRIBUTION**



(A)

$$q'_{max} = \frac{P}{B}\left(1 - \frac{6e}{B}\right)$$

$$q'_{min} = \frac{P}{B}\left(1 + \frac{6e}{B}\right)$$

(B)

$$q'_{max} = \frac{P}{B}\left(1 - \frac{6e}{B}\right)$$

$$q'_{min} = 0$$

(C)

$$q'_{max} = \frac{2P}{3B(0.5 - e/B)}$$

ALL PRESSURE DISTRIBUTIONS ARE BASED ON 1 FT. STRIP OF FOOTING. THESE EQUATIONS ARE RECOMMENDED FOR PRELIMINARY DESIGN.

FIGURE 6.5 – BEARING PRESSURE DISTRIBUTION

6.3.4 Abutment Stability

When determining the stability for an abutment, the following must be considered:

1) Uplift is prevented on the backside of the footing.
2) The resultant load, being the sum of the lateral earth pressure, abutment weight, and bridge weight, must lie inside the middle third of the base. Live load shall be included also.
3) Embankments shall be safeguarded against ground ruptures, by providing sufficient abutment depth below the ground surface in soils with low shear strength (shown in Figure 9.6 below).
4) The fill behind the abutment is properly drained.

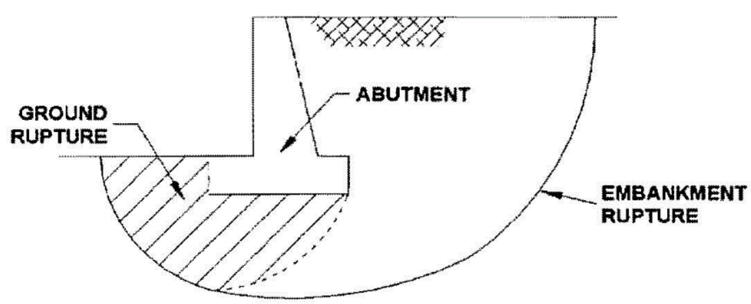


FIGURE 6.6 – STABILITY OF ABUTMENT

6.4 Clearances

PCJPB requires a minimum of 10 feet horizontal clearance from the centerline of track to the interior face of wall of the retaining structure. The absolute minimum horizontal clearance shall be per California PUC GO 26-D requirements.

Preferred vertical clearances shall be at least 25 feet 6 inches above top of rail. Minimum vertical clearance shall be 24 feet 6 inches with an absolute minimum vertical clearance of 23 feet 6 inches allowed only under special circumstances. Attention shall be given to overhead utilities that may be relocated during construction, if located within PCJPB Right-of-Way; there are special CPUC vertical clearance requirements for overhead utility lines (CPUC GO 95).

Right-of-Way fences shall be a minimum of 6 feet in height. Retaining walls joining fencing shall provide for continuous protection of the Right-of-Way by the 6 foot height limit.

6.5 Special Provisions

6.5.1 Railroad Electrification

PCJPB/Caltrain is currently under construction for electrification (25 kV AC 60 Hz) of the corridor. All structures shall provide for a future installation of an overhead catenary system. Space for catenary support poles along the PCJPB Right-of-Way and on structures shall be provided. Overhead vertical clearance and horizontal clearance shall comply with the following references:

CPUC GO 26-D and GO95, NESC, NEC, AREMA Chapters 28 and 33, and CPUC Resolution SED-2 dated November 10, 2016.

All concrete retaining walls shall be detailed to mitigate the effects of stray current corrosion of steel reinforcing, prestressing elements, and other steel components. This will require that electrical continuity be provided between all steel elements within each concrete structural component with provisions to connect the components to a central location. The corrosion control system will be installed at the time of future electrification with connections made to the central location. Comply with CPUC GO 95, AREMA Chapter 33, IEEE, NESC, and NEC provisions for stray current.

6.5.2 Utilities

Utility lines shall not be attached to new retaining structures, except for utilities (i.e., signal lines) that are required for the operation of the railroad. Existing or future fiber optic lines shall be placed underground and away from retaining walls. In most cases relocation of existing utilities will be provided by the owners of the utility. In some instances when the utility line may have to be located on the structure, PCJPB will handle these on a case by case basis. Utility locations need to be verified by qualified land surveyors and by information from respective public utility agencies.

6.5.3 Drainage

Retaining wall drainage shall be provided for and directed away from the track roadbed and away from the back of the wall. Water shall be removed preferably by horizontal drainpipes (larger than 8-inch diameter) or by weep holes (larger than 6-inch diameter spaced at maximum 10 feet apart). Drainage patterns for the entire Right- of-Way shall be considered for the project with all drainage directed away from the tracks and to an appropriate outfall.

6.5.4 Mechanical Stabilized Embankments (MSE)

MSE walls shall follow the requirements of AREMA – Chapter 8, Part 7. In addition to the AREMA guidelines the following provisions shall be incorporated into the design:

- The actual applied bearing pressures under the stabilized mass for each reinforcement length shall be clearly indicated on the design drawings.
- Passive pressure in front of the wall mass shall be assumed to be zero for design purposes.
- Due consideration shall be given to the placement of utilities along the railroad within the area of the MSE structure. This can be done by limiting the placement of the MSE reinforcements to a zone below the potential depth of trenching (4 to 6 feet) or by planning ahead and pre installing conduits to allow for future utility installations.
- Calculations for stresses and factors of safety shall be based on assumed conditions at the end of the design life.
- The design life shall be 100 years for MSE structures unless otherwise directed by the PCJPB.

(a) MSE that use Metallic Reinforcing Strips

In the case of MSE that use metallic reinforcing strips the following provisions apply. For determination of the allowable reinforcement tension, the following metal loss rates shall be used:

- Zinc (first 2 years): 15 microns/year/side
- Zinc (subsequent years to depletion): 4 microns/year/side
- Carbon Steel (after depletion of zinc): 12 microns/year/side
- Carbon Steel (from 75 to 100 years): 7 microns/year/side

For maximum allowable tensile stress in steel reinforcements and connections including tie strips, F_t at the end of the service life, shall conform to the following:

- Linear Reinforcements (strips)
  - $F_t = 0.55\ F_y$ at reduced gross cross section
  - $F_t = 0.50\ F_u$ at net section of bolted connections

- Bar Mats and Welded Wire Meshes
  - $F_t = 0.48\ F_y$ all sections

It is critical that the longitudinal and transverse wires (bars) be of the same size on any given reinforcement element. F_y used for design shall not exceed 65 ksi. The maximum allowable tension in the reinforcements shall consider any reductions in cross sectional area of reinforcements due to punching, corrosion losses, and shall not exceed 50% of the pullout capacity of the connection devices embedded in the facing panels.

(b) MSE Structural Backfill

The select granular backfill material used in MSE structures shall be reasonably free from organic and otherwise deleterious materials and shall conform to the following gradation limits:

Sieve Size	Percent Passing
4 inches	100
3 inches	75 to 100
No. 200	0 to 15

In addition the backfill shall conform to all of the following requirements:

1. The plasticity index (P.I.), as determined by AASHTO T-90, shall not exceed 6.

2. Soundness – The material shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles, as determined by AASHTO T-104.

3. Electrochemical Requirements – The backfill material shall conform to the following electrochemical requirements:

Property	Requirement	Test Method
Resistivity	Minimum 3,000 ohm-cm, at 100% saturation	ASTM G-57-78
pH	Acceptable Range 5 to 10	ASTM G-51-77
Chlorides	Maximum 100 ppm	ASTM D-512-88
Sulfates	Maximum 200 ppm	ASTM D-516-88

All testing reports and a Certification of Compliance certifying that the selected granular backfill material meets these requirements.

4. Backfill shall be compacted to 95% of the maximum density as determined by AASHTO T-99, Method C or D (with oversize correction).

(c) Corrosion

Special attention shall be given to the prevention of corrosion in addition to the items listed above especially if metal strips are used. The MSE wall shall be provided with inspection elements for removal and verification of the rate of corrosion occurring over the life of the wall. Inspection elements are ·to be clearly and permanently marked on the wall in the facing panels with the following intervals for inspection; 5, 10, 25, 50, 75, and 100-year time periods with a separate inspection element for each year.

Since the railroad roadbeds have a relatively high permeability the MSE reinforcements can be exposed to precipitation and potentially corrosive substances infiltrating through the roadbed. This issue can be resolved by incorporating an impermeable layer connected to lateral drains beneath the sub-ballast but above the level of the reinforcements or strips.

Stray currents from railroad electrification are also of concern to the corrosion of metal reinforcements. Research and studies have shown that there may be a risk of accelerated corrosion for systems that utilize direct current. However, on alternating current systems this is not apparent. Other studies have shown that galvanized reinforcements have been used where direct current systems are present with stray current corrosion being negligible for the following reasons:

1) The individual reinforcement strips are short in comparison to the rails. Thus the electrical potential along the strips is minuscule for collecting stray currents.

2) The reinforcements are electrically discontinuous since the soil reinforcements are discrete linear strips.

3) Reinforcing strips are generally placed perpendicular to the direction of the running rails and the return current flow. This orientation offers the greatest resistance to stray current collection.

4) Use of a high resistivity select granular backfill reduces the tendency for stray currents to flow within the MSE volume.

It is recommended that a firm or expert in the area of corrosion and stray currents be retained to make site-specific recommendations where MSE structures are to be utilized.

6.5.5 Crib Walls

Crib walls is a gravity structure consisting of rigid open shapes filled with granular material. Types of crib walls include:

- **Concrete crib walls –** Max design height of 50 feet. Suitable for coastal areas and higher elevations.

  The maximum area of reinforcement shall be 0.9% of its gross rectangular cross-sectional area. Minimum compressive strength of 4,250 psi for concrete at 28 days shall be used.

- **Steel crib walls –** Max design height of 36 feet. Suitable for difficult installation sites due to its lightweight and easy transportation.

  Base metal used shall meet chemical composition and zinc composition.

- **Timber crib walls –** Max design height of 22 feet. Aesthetically compatible with rural environment and comparable service life to that of concrete and steel.

For overturning moments, the wall section may be assumed as a rectangle with total height of the crib wall, and depth equal to the distance between the two outside faces of the crib structure.

Provisions for drainage shall be made behind the wall.

Crib walls shall be designed to withstand a differential deflection of 0.015L, where L is the length of the cell measured along the face of the wall.

Other requirements for crib wall design shall be in accordance with AREMA, Chapter 8, Part 6.

Crib walls shall be handled carefully to avoid any damage due to shock or impact. Cracked or damaged members shall be replaced and removed during erection. The interior of the wall shall be filled immediately before the installation of other tier units.

6.5.6 Bulkhead/Cantilever Systems

Bulkhead systems include cantilever pile, sheet pile, tieback anchored pile, or soldier pile walls. Bulkheads are used where excavation is limited by traffic, utilities, or right of way restrictions. Costs of bulkhead walls depend on specified design requirements, site restraints, and aesthetic considerations.

Provisions in AREMA, Chapter 8 for Sheet Pile Bulkheads are based on service load design only. Sheet pile bulkheads shall be designed to be flexible structures that mobilize full active earth pressure and a portion of the passive pressure. A movement of $0.001H_f$ to $0.002H_f$ is needed to develop full active pressure in anchored bulkheads, where H_t is the height of the sheet pile above the soil level.

(a) Loading

Bulkheads may be subjected to, but not limited to, the following loads:

- Temporary loads
  - Construction equipment
  - Construction materials
  - Hydrostatic pressure

- Permanent loads
  - Future grading and paving
  - Railroads or highways
  - Structures
  - Material storage piles
  - Snow and earthquake

6.5.7 Tieback Anchored Piles

Tieback walls may consist of: sheet piles with horizontal wales, vertical soldier piles with timber or concrete lagging, reinforced concrete slurry or shotcrete walls.

(a) Loading

Tieback anchor piles shall be designed as beams subjected to the loading of the appropriate lateral pressure diagram. Horizontal wales shall be designed as simple beams loaded by the reactions from the lateral pressure diagram. The level of soil excavation shall provide sufficient bearing capacity to support the tieback force.

Tiebacks form a prestressing system anchored in a drilled hole filled with PCC grout. The prestressing steel design for tieback anchor piles is at a maximum stress of $0.55f_{p\mu}$ and jacking load of $0.75f_{p\mu}$, where $f_{p\mu}$ is the ultimate strength of prestressing steel in lbs/in^2. The designer shall develop the design force “T,” the force required to resist the design lateral earth pressure. The minimum factor of safety for the tieback force shall be 1.25.

(b) Construction Sequencing

Proper installation of tiebacks is critical to prevent overstressing of each anchor. Over-excavation is a cause of overstressed tiebacks. Precautions shall be made to specify the maximum allowable level of excavation.

When more than two levels of tiebacks are to be used, PCJPB recommends that the lower tieback be lengthened to avoid forming a circular slip plane and causing settlement failure.

Major failures have been known to occur if lagging is not placed simultaneously with the excavation progress.

6.5.8 Conventional Retaining Walls

The base of a retaining wall or abutment footing shall be embedded at least 3 feet below the ground surface in front of the wall face.

The unit shear stress for a horizontal shear key shall not exceed $0.25f'_c$.

Vertical keyed wall expansion joints may be spaced at a maximum of 60 feet apart with appropriate waterproofing.

The minimum compressive strength of concrete to be used for retaining walls shall be 3,600 psi at 28 days.

Reinforcement for retaining walls shall meet the following requirements:

- ASTM Standard A615 Grade 60, or
- ASTM Standard A706, or
- Welded steel wire fabric ASTM Standard A497.

A maximum allowable tensile stress of 24,000 psi shall be used in service load design for the reinforcing steel.

CHAPTER 7

STRUCTURE DESIGN SUBMITTAL PROCEDURES

CHAPTER 7: STRUCTURE DESIGN SUBMITTAL PROCEDURES

7.1 General

Prior to the submittal of any project involving PCJPB facilities, it is important that the agency or applicant be familiar with the guidelines and operations of the PCJPB. Any issues with the design guidelines shall be addressed prior to submitting any documents to the PCJPB. Submittals for design and construction of structures and grade separations projects on PCJPB property shall be coordinated and submitted to the PCJPB. To expedite reviews, submittals must be complete, clearly explained and assembled in an orderly manner. Plan reviews will either be performed by PCJPB staff, an outside consultant, or a combination of both.

All engineering review work required of the PCJPB by a sponsoring agency or firm for design and construction documents for structures, grade separations, construction submittals, falsework, and shoring plans, etc., shall be paid for by the applicant or sponsoring agency, through an agreement with the PCJPB.

All plans for structures on PCJPB Right-of-Way or that affect PCJPB operations shall be approved by the PCJPB staff before construction begins.

The projects covered under these guidelines will require certain agreements, permits, right-of-entry permits, and CPUC applications be prepared, submitted and approved prior to construction.

The California Public Utilities Commission must approve all alterations, relocations, or detours that affect track alignment, grade, or the addition of tracks at grade crossings and grade separations.

7.2 Design Submittals – Master List

Project plan documents submitted for review shall include the following submittals to the PCJPB:

- Conceptual Design Level – Approximately 10 to 15% design concept
- Preliminary Design Level – Approximately a 35% design completion
- In-Progress Design Level – 65% design completion
- Pre-Final Design Level – 95% design completion
- Final Design Level – 100 % design completion
- Camera Ready Level or Bid Documents
- Bid Period Addendum Documents (required if a PCJPB project)
- Conformed Documents for Construction
- As-Built Documents

7.2.1 Conceptual Design Level – Approximately 10 to 15% Design Concept

Conceptual design level shall be presented in a preliminary meeting with the PCJPB to verify form, function and configuration of the project. Typical submittal item for discussion shall be the minimum of a layout plan.

7.2.2 Preliminary Design Level – Approximately a 35% Design Completion

Preliminary design plan submittal shall be comprised of the layout plans, (including detailed geometry, profiles, typical sections, track and lane configuration), bridge type selection report, and with critical construction issues raised. Preliminary cost estimate to be included as well.

Specific items to be included shall be:

- Plan view of proposed structure and location of all existing facilities and utilities within the operating Right-of-Way. Plan view to indicate span lengths, alignment, skew angle of abutments and piers, site drainage, etc.
- Elevation view indicating the abutment and pier elevations, track elevation to top of rail for existing and proposed, minimum vertical clearance and location, footing elevations, types of footings, locations of utilities and relocations.
- Typical superstructure cross section showing deck and pier outline, horizontal and vertical dimensions of deck structure, rail and ballast structure, waterproofing, deck drainage, track spacing, horizontal and vertical clearances, etc.
- The existing and proposed track profile shall be shown at least 1,000 feet beyond the proposed structure in each direction.
- The existing and proposed track and roadway alignment shall be shown. Any proposed shoofly alignment shall be shown. The alignment design data shall be provided.
- General notes shall be shown to indicate the design criteria, construction materials, and construction sequencing. Preliminary plans on construction staging shall be provided.
- Plans shall identify the locations of existing and relocated utilities. It is very likely that there are fiber optic cables buried along the PCJPB Right-of-Way. The identification of the location of the fiber optic cables and there relocation or protection shall be consider part of the project and be addressed in the plans and specifications.
- The structure general plan shall show the location of the shoofly if required and indicate the footprint of the structure and its relation to the shoofly. Minimum clearance distances shall be shown.
- Preliminary hydrology and hydraulic (H&H) reports shall be developed. High and low flow levels shall be determined. Rainfall intensities investigated. Issues on bridge scour and protection are to be investigated.

Allow a minimum of 3 weeks review time.

7.2.3 In-Progress Design Level – 65% Design Completion

Intermediate design submittal will typically be comprised of the final layouts, profiles and typical sections, in-progress drainage plans, construction staging and traffic handling plans and impacts, utility layout plans, bridge general plans and foundation plans, Right-of-Way and permit requirements. Include a detailed outline of specifications and an intermediate cost estimate of the project.

Specific items shall include the following if they apply:

- Design of structure including superstructure and substructure
- Structure details
- Bearing details
- Deck and waterproofing details
- Geotechnical reports and recommendations
- Structural calculations
- Drainage layout and report
- Utility relocations
- Complete track profile and alignment information and supporting data; also, include shoofly date if applicable
- Final hydrology and hydraulic report
- Construction sequence and staging; indicate locations of shoring, falsework, and temporary facilities that may affect train operations

This submittal may or may not be sent to external review depending on the complexity of the project.

Allow 4 weeks for review.

7.2.4 Pre-Final Design Level – 95% Design Completion

Pre-final design submittal shall include complete plans, specifications, and quantities that are essentially complete. Plans shall include engineering seals if required for agency review. Intended use is for formal review, including internal and external agency comments and approvals. At this level an independent check shall be performed.

Specific items shall include the following:

- Completed structural calculations with revisions from the 65% submittal
- Complete plans to date
- Complete project specifications
- Complete reports as required for project
- Updated cost estimate
- Documented independent check

Allow 5 weeks for review.

7.2.5 Final Design Level – 100 % Design Completion

Final design submittal shall include all plans, specifications, and cost estimates. Verify that all reviewers' comments have been addressed and incorporated and that the submittal is ready for printing.

Submittal of final contract documents with original signatures. If it is not a PCJPB contract, submit a copy of the bid documents.

Specific items of the submittal shall include the following:

- Final plans and specifications signed and sealed by a register professional engineer in the state of California.

- Final structural calculations signed and sealed by a register professional engineer in the state of California. The designer and the checker shall sign each calculation sheet.

- Final estimates with quantity calculations

- Final reports and data documents signed and sealed by a register professional engineer in the state of California.

Allow 3 weeks for final review and approval.

7.2.6 Bid Documents

Camera-Ready Level or Bid Documents (required if a PCJPB project). Submittal of final contract documents with original signatures, ready for printing.

7.2.7 Bid Period Addendum Documents

Engineers' updates including addenda and changes to the plans, specifications, and estimates and responses to questions required during the bid period (required if a PCJPB project).

7.2.8 Conformed Contract Documents for Construction

Complete set of Plans, Specifications, and Estimate with bid pricing, including all addendum's and changes made during the bid period. This is the record contract documents to be used during construction.

7.2.9 As-Built Documents

(See Construction Submittals)

7.3 Submittal Quantities

7.3.1 Conceptual Design Level – Approximately 10 to 15% Design Concept

- One full-size print and two half-size prints
- Two sets of concept reports

7.3.2 Preliminary Design Level – Approximately a 35% Design Completion

- One full-size print and two half-size prints
- Two sets of preliminary reports, including hydrology and hydraulic, geotechnical, and final bridge type selection reports
- Two sets of cost estimate

7.3.3 In-Progress Design Level – 65% Design Completion

- One full-size print and two half-size prints
- Two sets of preliminary reports, including geotechnical reports if revised

- Two sets of final hydrology and hydraulic report
- Two sets of response to 30% comments
- Two sets of specifications outline, estimate, and schedule
- Two sets of preliminary calculations
- Two sets of permit requirements, table, and status of each parcel
- Two sets of utility impacts table
- Two sets of Right-of-Way requirements table and final easements/ROW descriptions.

7.3.4 Pre-Final Design Level – 95% Design Completion

- One full-size print, two half-size prints, and one reproducible half-size print
- Two sets of response to 60% comments
- Two sets of specifications, cost estimate, and schedule
- Two sets of design calculations including computer printouts; all computer printouts to be supplemented with notes explaining input and output information
- Two sets of permit requirements, table, and status of each parcel
- Two sets of updated utility impacts table
- Two sets of Right-of-Way requirements table and final easements/ROW descriptions if revised
- Two sets of independent check documents

7.3.5 Final Design Level – 100% Design Completion

- One full-size print, two half-size prints, and one reproducible half-size print
- Two sets of response to 85% comments
- Two sets of specifications, cost estimate, and schedule
- Two sets of final reports
- Two sets of revised design calculations

7.3.6 Bid Documents

- Two sets of responses to final design review
- Camera-Ready Bid Documents
- Original full size full-size sealed plans, one reproducible full-size print and one reproducible half-size print.
- Electronic data files for drawings and specifications
- Original sealed specifications
- Final engineers cost estimate and schedule

7.3.7 Bid Period Addendum Documents

- Bid period addendum documents (required if a PCJPB project)
- Responses to contractor’s questions
- Updated engineers estimate and revisions to plans and specifications as required

7.3.8 Conformed Contract Documents for Construction

- Conformed plans and specifications documents for construction

7.3.9 As-Built Documents

- As-Built Documents (See Construction Submittals)

  All submittals shall include information deemed necessary to clarify the design, such as manufacture's catalogues and brochures.

  All plans, reports, calculations, and data listed above shall be submitted to the PCJPB. The designer shall certify that all the contract documents have been verified and checked in accordance with the Quality Assurance Plans of the firm.

7.4 Construction Requirements for Structures

7.4.1 Construction Management

For construction of grade separation underpass structures an experienced Construction Management Team (CM Team) will be required. The team who has been approved by the sponsoring agency and the PCJPB will typically consist of the following, depending on the project size:

- Project Manager
- Resident Engineer
- Construction Inspector

The members of this team will be required to have obtained PCJPB Safety Program for Roadway Worker Protection training and comply with the FRA Bridge Worker Safety Standards.

The team shall be responsible for notifying the PCJPB of significant milestones during the construction process. The following list represents some of the significant construction stages that the PCJPB needs to be present:

- Preconstruction meeting
- Shoofly work for acceptance prior to being placed in service
- Reinforcement and concrete placement
- Erection of steel and precast concrete
- Post tensioning
- Waterproofing and protection board placement for acceptance prior to ballast placement
- Final inspection and acceptance of the bridge structure
- During the course of the construction, PCJPB will make other periodic site visits to verify progress and inspect the work site.

Costs for the Construction Management Team shall be borne by the sponsoring agency.

7.4.2 Construction Submittals

During the construction process for grade separations the PCJPB will require the review of certain contractor submittals in order to avoid interruption to PCJPB operations and to verify the quality of the construction. Some of these submittals are listed below:

- Construction schedule with periodic update

- Site-specific workplans (SSWP) pertaining to PCJPB Right-of-Way
- Staging plans involving PCJPB operations
- Shoring plans
- Falsework installation and removal plans
- Temporary drainage facilities
- Contractors or Sub certifications for members being prefabricated
- Material certifications
- Welder certifications
- Shop drawings for steel and concrete members
- Bearings for the structure
- Concrete mix design
- Rebar and Strand certifications and shop drawings
- 28-day concrete strength tests
- Waterproofing system and protection board material and certifications
- Structural steel certifications for fracture critical members and test results
- Foundation construction reports for pile driving, drill shaft construction, or bearing pressure test reports.
- Pile driving hammer information
- Any other shop drawings or working drawings pertaining to the structure on PCJPB Right-of-Way.

The CM Team and designer shall review all contractor submittals. Any issues shall be prior to being submitting to the PCJPB for review and approval. No work shall be performed inside the PCJPB Right-of-Way without prior review and approval. All site-specific work plans submitted by the contractor that affect the operations of the railroad shall include a contingency plan for putting the rail operation system back in service in case of an emergency. The contingency plan shall address the stages of construction and may require redundant equipment and personnel be on call to satisfy this requirement.

7.4.3 As-Built Submittal

At the completion of the projects construction, after final acceptance of the project, the PCJPB shall receive the project As-Built documents. As-Built documents shall consist of the following and be signed and sealed by the California Registered Professional Engineer:

- Final As-Built project plans

- Final shop drawings for prefabricated components (structural steel, precast concrete, bearings, etc., and the test reports for these items)

- Final project specifications

As-Built plan submittal shall be full size drawings and in electronic file format.

Peninsula Corridor Joint Powers Board
UNDERPASS GRADE SEPARATION DATA SHEET

1. Location: ______________ ______________ ______________
   CITY COUNTY STATE

2. Distance from nearest PCJPB Milepost to centerline of bridge: ______________

3. Description of Project: ______________

4. Utilities on Railroad Property:

COMPANY NAME	RELOCATION Required?	CONTACT PERSON

5. List the at-grade crossings that will be eliminated by the construction of this grade separation.

DOT #	MILEPOST	SIGNALS

6. How many spans are proposed: ______________

7. Offset of temporary detour/shoofly alignment: ______________

8. Temporary detour alignment
   On Embankment, Trestle, or Both: ______________

Peninsula Corridor Joint Powers Board
UNDERPASS GRADE SEPARATION DATA SHEET (PAGE 2)

9. DRAINAGE:
Describe how drainage from track roadbed is handled: ____________________

____

____

____

____

Describe how drainage from the bridge is handled: ____________________

____

____

____

____

10. Scheduled Bid Date: ____________________

ALL INFORMATION ON THIS DATA SHEET TO BE FURNISHED BY THE SUBMITTING AGENCY TO THE PENINSULA CORRIDOR JOINT POWERS BOARD.

Peninsula Corridor Joint Powers Board
OVERHEAD GRADE SEPARATION DATA SHEET

1. **Location:** ____________ ____________ ____________
   CITY COUNTY STATE

2. **PCJPB Milepost to Centerline of Bridge:** ____________

3. **Description of Project:** ____________

4. **Utilities on Railroad Property:**

COMPANY NAME	RELOCATION Required?	CONTACT PERSON

5. **List the at-grade crossings that will be eliminated by the construction of this grade separation.**

DOT #	MILEPOST	SIGNALS

6. **Minimum horizontal clearance from centerline of the nearest track to face of Pier:**

   A. Proposed: ____________ **B. Existing (if applicable):** ____________

7. **Minimum vertical clearance above tope of high rail:**

   A. Proposed: ____________ **B. Existing (if applicable):** ____________

Peninsula Corridor Joint Powers Board
OVERHEAD GRADE SEPARATION DATA SHEET (PAGE 2)

8. List Piers where crash walls are provided:

PIER	DISTANCE FROM CENTERLINE OF TRACK

9. Describe how drainage from track roadbed is handled: ______________

10. Describe how drainage from overhead grade separation is handled: ______________

11. List piers where shoring is required to protect track: ______________

12. Scheduled Bid Date: ______________

ALL INFORMATION ON THIS DATA SHEET TO BE FURNISHED BY THE SUBMITTING AGENCY TO THE PENINSULA CORRIDOR JOINT POWERS BOARD.

Caltrain Peninsula Corridor Joint Powers Board
OVERHEAD SUBMITTAL CHECKLIST

PRELIMINARY PLAN SUBMITTAL CHECKLIST

Highway/Street Name: ______________________
Location (City & State): ______________________
County: ______________________
Project No.: ______________________
Date: ______________________

FILE:

GRADE SEPARATION:

STATE:	LOCATION:	
STREET/HWY:		
RTE:	M.P.	SUB:
DOT No.:		

Item	Required Information	Min. Required	As Submitted	Railroad Remarks A = Approved R = Rejected	
				A/R	
	Abutment				
1	Horizontal Clearance (Left) (CL to face)	25'-0"			
2	Horizontal Clearance (Right) (CL to face)	25'-0"			
3	Vertical Clearance (from Top of Rail)	25'-6"			
4	Horizontal Clearance to footing from CL	25'-0"			
5	Depth top of footing below base of rail	6'-0"			
6	Pier Protection wall required for <25'	25'-0"			
7	Shoring required (CL to nearest Pt.)	8'-6"			
	Pier/ Bent #				
1	Horizontal Clearance (Left) (CL to face)	25'-0"			
2	Horizontal Clearance (Right) (CL to face)	25'-0"			
3	Vertical Clearance (from Top of Rail)	25'-6"			
4	Horizontal Clearance to footing from CL	25'-0"			
5	Depth top of footing below base of rail	6'-0"			
6	Pier Protection wall required for <25'	25'-0"			
7	Shoring required (CL to nearest Pt.)	8'-6"			
	Pier/Bent #				
1	Horizontal Clearance (Left) (CL to face)	25'-0"			
2	Horizontal Clearance (Right) (CL to face)	25'-0"			
3	Vertical Clearance (from Top of Rail)	25'-6"			
4	Horizontal Clearance to footing from CL	25'-0"			
5	Depth top of footing below base of rail	6'-0"			
6	Pier Protection wall required for <25'	25'-0"			
7	Shoring required (CL to nearest Pt.)	8'-6"			
	Abutment				
1	Horizontal Clearance (Left) (CL to face)	25'-0"			
2	Horizontal Clearance (Right) (CL to face)	25'-0"			
3	Vertical Clearance (from Top of Rail)	25'-6"			
4	Horizontal Clearance to footing from CL	25'-0"			
5	Depth top of footing below base of rail	6'-0"			
6	Pier Protection wall required for <25'	25'-0"			
7	Shoring required (CL to nearest Pt.)	8'-6"			

COMMENTARY

COMMENTARY

The purpose of this commentary is to furnish technical explanations, clarifications, and examples for the various Chapters in the Standards for Design and Maintenance of Structures. The numbering of the sections in this Commentary corresponds to the numbering in the Chapters.

C-1. General

C-1.1 Application of Guidelines

C-1.2 Compatibility

C-1.3 Design Guidelines, Codes, Manuals, Standards, and Specifications

C-1.4 General Guidelines for Grade Separations

In regard to PCJPB's preference for overheads, the following is an excerpt from the Caltrans Highway Design Manual, Section 208.9, Railroad Underpasses and Overheads.

> "Generally, it is desirable to construct overheads rather than underpasses whenever it is necessary for a highway and railroad to cross. Railroads should be carried over highways only when there is no other reasonable alternative.
>
> Some undesirable features of underpasses are:
>
> a) They create bottlenecks for railroad operations.
>
> b) It is difficult to widen the highway.
>
> c) Pumping plants are often required to drain the highway.
>
> d) They are likely to lead to cost participation controversies for initial and future construction.
>
> e) Shooflies (temporary tracks) are generally required during construction.
>
> f) Railroads are concerned about the structure maintenance and liability costs they incur.
>
> Advantages of overheads are:
>
> a) Railroads can use most of their right of way for maintenance.
>
> b) Overheads can be widened at a relatively low cost and with little difficulty.
>
> c) Less damage may be incurred in the event of a derailment.

d) Agreements for design and maintenance can be reached more easily with railroads.

e) Initial costs are generally lower.

The State, the railroads, and the public in general can benefit from the construction of an overhead structure rather than an underpass."

C-2. Design Guideline for RAILROAD BRIDGES and UNDERPASSES

C-2.1 General Requirements

C-2.2 Superstructure Selection Type

Interruptions in service are much more critical for railroads than for highways (alternate routes are not always available); therefore, constructibility and maintainability without interruption to rail traffic are crucial in the design and construction for rail structures. The duration of construction windows required during rail operations will play a significant part in deciding which type of structure to use for each location. The construction staging and the windows that are required during construction should be discussed early with the PCJPB to determine this and other constraints that will control the design. Also, the type of structure will be affected if a shoofly track or temporary structure is available during construction. Another factor to consider is if the structure can be staged to allow for rail traffic to resume on half or a portion of the structure, prior to the full bridge being completed.

Construction must be planned to minimize delay or disruption of PCJPB train schedules and operations. Work that will affect operation will require work during night periods, during off-peak commute hours, or during weekends. Provisions for shooflys, temporary structures, false work and shoring must minimize the extent and severity of slow orders on operations.

Ballast deck structures require less maintenance than open deck or direct fixation type structures. For this reason ballast deck structures are preferred.

Simple span superstructure construction is fast and lends itself to rapid erection, and therefore is preferred. This is a major consideration on high traffic lines such as the PCJPB corridor which can ill afford shutdowns in its operations. Continuous span construction takes longer to construct and will require greater diligence during construction in the inspection and monitoring of bridge construction to maintain quality. Continuous span structures are also more difficult to replace in emergencies than simple spans.

Trough type superstructures are not readily repairable if damaged, and can result in an extended period of time to replace. This out of service time to repair or replace a structure can have an adverse effect on PCJPB operations and its service to their customers.

Aesthetics of the structure will contribute to the selection of the superstructure; however it shouldn't control the decision of the final bridge type. All of the structures on this list can, with some thought, be constructed so that they are aesthetically pleasing. The use of fascia panels and other architectural treatments can be used to meet the public's aesthetic requirements. However, care must be taken when applying these treatments in that they do not interfere with the inspection and maintenance of the structure.

C-2.2.1 List of Preferred Structure Type

Generally, deck-type structures are preferred over through-type structures, because deck-type structures are less likely to be damaged in a train derailment or by dragging rail equipment.

Structures of steel and precast concrete are preferable because they are fabricated off site and the quality is shop controlled. Also, these types can be constructed rapidly which is an advantage when working around train operations. Steel structures are also preferred because they are readily repairable. Precast concrete structures are close in ranking to steel structures because of their low maintenance, although the use of weathering steel makes this less of an issue. Cast-in-place girders and components, and post-tensioned structures are less desirable because of the additional construction time required on site and the need for diligent on-site inspection to assure quality control.

C-2.3 Structural Design Requirements

Differences between Railroad Bridges and Highway Bridges
(courtesy of AREMA Railway Structures Loading Seminar, Course Manual)

a) The live load to dead load ratio is much higher for a railroad bridge than for a similarly sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.

b) The design impact load on railroad bridges is higher than on highway structures.

c) Simple span structures are typically preferred over continuous structures for railroad bridges.

d) Interruptions in service are typically much more critical for railroads than for highway agencies. Therefore, constructability and maintainability without interruption to rail traffic are crucial for railroad bridges.

e) Since the track structure is supported by the bridge, the combination of track and bridge movement cannot exceed the tolerances in track standards. Interaction between the track and bridge should be considered in design detailing.

f) Seismic performance of highway and railroad bridges can vary significantly. Railroad bridges have performed well during seismic events.

g) Railroad bridge owners typically expect a longer service life from their structures than highway bridge owners expect from theirs.

h) Railroad bridges are typically designed to require as close to zero maintenance as possible.

Items for Designers to be Aware of Regarding Railroad Bridge Design
(courtesy of AREMA Railway Structures Loading Seminar, Course Manual)

a) For steel structures, pay close attention to fatigue design procedures.

b) Provide support perpendicular to track at end of skewed structures.

c) For concrete design, pay close attention to load factors and strength reduction factors. Note that reinforced concrete and prestressed concrete requirements have some differences. Also, note differences in the impact factor for reinforced concrete and prestressed concrete.

d) Remember the 0.25 sq. in./ft. minimum temperature and shrinkage reinforcing steel requirement for concrete structures.

e) AREMA does not allow service load tension in prestressed concrete members.

f) Provide member load tables on the drawings.

g) Be careful when using computer programs for railroad bridge design. Verify that the program is actually following AREMA recommended practice or the specific requirements of the railroad.

h) Note that AREMA does not use tension field design for steel girder webs.

i) Communicate with the railroad (PCJPB) about their specific requirements before designing their structures.

C-2.3.1 Layout

The commuter train operations along PCJPB corridor is a high traffic corridor and the scheduled trains have priority over construction projects during the normal workday week. Most of the construction work that is required to take place within 15 feet of the outside rail of the track or has the potential to come within this region (i.e., a tall crane that may topple into the region) will in most cases require a construction window (i.e., a period of time granted where there is protection from train traffic) or be required to be done during night time or weekend hours upon approval of the PCJPB.

Attachments of utility lines to the bridge are not allowed because they interfere with the maintenance of the structure. Items where utility attachments interfere with rail operations on structures include inspection, maintenance, and operational issues. These include: bridge inspection such as inadequate access to view bridge components, bridge maintenance such as when the bridge needs to be jacked or shored up for repair but the utility prevents it, painting of the structure, or a utility inhibits access to an areas that requires repair. Train Operations can be affected by an emergency of an outside agency's utility problem that is attached to a bridge, resulting in a track having to be shut down for repairs to a utility line.

Drainage and water is one of the main causes of track roadbed failure. All drainage shall be adequately provided for and directed away from the tracks.

Some of the track tolerances governed by 49 CFR – Part 315 Track Safety Standards are for line, profile, cross-level, and twist. Structures shall be designed such that they will not be a contributing factor to the loss of track tolerances.

The use of metal inner guard rails is used to minimize damage that that could occur when a train derailment occurs. Additional information on the details and limits of guard rail installation can be found in the AREMA Manual of Railway Engineering, Chapter 7, Part 3, Section 6.

C-2.3.2 Bridge Skew

Construction of skewed bridges can have a significant impact on the maintenance budget of the PCJPB therefore approval from the PCJPB is required when the design of a skewed bridge is required. The PCJPB may be able to suggest designs that will lessen the skew angle. Skewed bridges over crossings complicate the construction requiring greater detail, inspection, and a more detailed seismic analysis. Also, additional maintenance is required for a skewed bridge.

Abutments need to be squared-off to allow a full railroad tie to be supported. Partial support of a tie skewed across an abutment backwall will not be allowed. Problems can develop if a tie is only partially supported over the abutment. Ride quality will suffer and tie life will be shortened it the tie is not fully supported.

Track transitions are required to equalize the change in vertical stiffness that occurs when the track approach comes on to a structure. If no provisions are provided at transition regions, the track will require frequent maintenance. If this is neglected, it can deteriorate the rail alignment and surface at an accelerated rate. Low track approaches to a bridge will occur when the track transition is not properly accounted for, resulting in poor ride quality and a shortened track life.

C-2.3.3 Design Loads for Railroad Bridges

Various bridge dead loads are shown in the charts below. The PCJPB has included these values to indicate how the dead loads are established for design calculations. Designers shall generate independent calculations based on the specific conditions of the structure being designed. In no case shall the design load be less than that required by AREMA. The dead loads provided in the AREMA Manual are the minimum values to be used in design, however, if the actual loads are greater than the AREMA minimum requirements the actual load shall be used.

WEIGHT OF TYPICAL BALLAST DECK, SINGLE TRACK, 20-FOOT DECK

Item	Weight (lbs./LF of track)
RAIL (136 RE): 136 lbs/linear yd by 2 rails/track by 1 linear yd/3 linear ft.	91
INSIDE GUARD RAILS: 115 lbs/linear yd by 2 rails/track by 1 linear yd/3 linear ft.	77
TIES (Neglect, since included in ballast weight tor wood ties)	
TIE PLATES (7¾” by 14¾” for rail with 6” base): 24.32 lbs/plate by 1 tie/19.5” by 12”/ft. by 2 plates/tie	30
SPIKES (5/8” by 5/8” by 6” reinforced throat): 0.828 lbs/spike by 18 spikes/tie by 1 tie/19.5” by 12”/1 ft.	9
BALLAST* (assume 12” additional over time): Approximately 120 lbs/ft^3 by 30” depth by 1 ft./12” by 20 ft.	6,000
WATERPROOFING: Approximately 150 lbs/ft^3 by 0.75” depth by 1 ft./12” by 24 ft.	225
TOTAL WEIGHT:	**6,432 lbs/LF**

NOTES: Wood Ties weigh 237 lbs. for a 7-inch by 9-inch by 9-foot-long ties and are spaced at 19½ inches o.c. Concrete Ties weigh 630 lbs. for 8-foot-3-inch-long ties and are spaced at 24 inches o.c. 120 lbs/ft^3, used in ballast calculation, includes the weight of timber ties. If concrete ties are used, the weight of concrete ties and ballast shall be figured separately.

Concrete ties will also transfer higher live loads and impact load through the ballast to the bridge deck structure than wood ties. This shall be considered in the determination of the loads to be applied to the structure during design.

WEIGHT OF RAIL, INSIDE GUARD RAILS AND RAIL FASTENINGS FOR TYPICAL DECK

Item	Weight (lbs./LF of track)
RAIL (136 RE): 136 lbs/linear yard by 2 rails/track by 1 linear yd/3 linear feet	91
INSIDE GUARD RAILS: 115 lbs/linear yd by 2 rails/track by 1 linear yd/3 linear feet	77
TIE PLATES (7¾” by 14¾” for rail with 6” base): 24.32 lbs/plate by 1 tie/19.5” by 12”/ft. by 2 plates/tie	30
SPIKES (5/8” by 5/8” by 6” reinforced throat): 0.828 lbs/spike by 18 spikes/tie by 1 tie/19.5” by 12”/1 ft.	9

TOTAL WEIGHT:	207 lbs/LF

WEIGHT OF RAILS, INSIDE GUARD RAILS, TIES, GUARD TIMBERS, AND FASTENINGS FOR TYPICAL OPEN DECK (WALKWAY NOT INCLUDED)

Item	Weight (lbs./LF of track)
RAIL (136 RE): 136 lbs/linear yd by 2 rails/track by 1 linear yd/3 linear ft.	91
INSIDE GUARD RAILS: 115 lbs/linear yd by 2 rails/track by 1 linear yd/3 linear ft.	77
TIES (10” by 10” by 10 ft. bridge ties): 10” by 10” by 10 ft. by 1 ft^2/144 in^2 by 60 lbs/ft^3 by 1 tie/14” by 12”/1 ft.	357
GUARD TIMBERS (4 in. by B in.): 4” by 8” by 1 ft. by 1 ft^2/144 in^2 by 60 lbs/ft^3 by 2 guard timbers/ft.	27
TIE PLATES (7%” by 14%” for rail with 6” base): 24.32 lbs/plate by 1 tie/14” by 12”/1 ft. by 2 plates/tie	42
SPIKES (5/8” by 5/8” by 6” reinforced throat): 0.828 lbs/spike by 18 spikes/tie by 1 tie/14” by 12”/1 ft.	13
MISCELLANEOUS FASTENINGS (Hook Bolts an Lag Bolts) Approx. (2.25 lbs/hook bolt + 1.25 lbs/lag screw) by 2 bolts/tie by 1 tie/14” by 12”/Ht	6
TOTAL WEIGHT:	**613 lbs/LF**

The design Cooper E-80 live “load of is a bridge loading, thus the application of this loading to other structures requires special judgement on the part of the engineer. It should be noted that the Cooper train loading does not reflect the actual loading of any equipment in current revenue service on the rail lines in the United States. Therefore, included in the Appendix are the diagrams for the equipment used along the PCJPB corridor for passenger service, and some of the freight equipment. The diagrams give the equivalent Cooper E loading generated by equipment for various span lengths. Upon review of the equipment diagrams, it will become apparent that the loading of this equipment is less than the Cooper E-80 loading that the PCJPB requires for bridge design. The PCJPB requires the use of Cooper E-80 loading because this corridor is required to be able to accept freight service (Union Pacific Railroad has trackage rights), thus their heavier equipment may approach the E-80 loading conditions especially on some shorter span bridges. The E-80 loading is also specified because in the railroad industry heavier locomotives and heavier cars are the trend. Prior to 1967 the Cooper loading used was E-72. It should also be noted that the PCJPB corridor has historically been designated as satisfactory for 315,000-pound (125-ton car) cars. Generally for railroad service, the Association of American Railroads (AAR) has designated three types of rail cars for line weight clearance, 263,000-pound cars (100-ton cars), 286,000-pound cars (110-ton cars) and the 315,000-pound car (125-ton car). The 315,000-pound car has an axle loading of 78,750 pounds. The PCJPB corridor from San Francisco to Santa Clara, which has Union Pacific Railroad trackage rights and a weight limit of 315,000 pounds, and from Santa Clara to Lick where PCJPB shares the tracks of the Coast Route with the Union Pacific Railroad, also has 315,000 pound weight limit. The designer should be aware that there are also on-track maintenance-of- way vehicles (i.e., ballast cars, locomotive cranes, and wrecking cranes, etc.) that may have an axle loading greater than the cars shown in the Appendix. There are also specialty cars that shippers use to haul heavy loads that have non-standard axle spacing and axle loads that can cause moments and shears in structures greater than the E-80 loading condition. Additionally, the standard railroad car can be overloaded which is most likely to occur in car types that have bulk lading (gravel, grain, and coal commodities). The magnitudes of overloads may not be significant enough to cause structural problems other than the variations in stress levels, which can result a faster fatigue damage rate.

C-2.4 Clearances

C-2.4.1 Vertical Clearances

C-2.4.2 Horizontal Clearances

The 10-foot recommended horizontal clearance on structures has been established by the PCJPB to provide for additional walkway room in the case of an emergency where a train was stopped over a structure. It will provide for access of emergency personnel and the evacuation of passengers safely.

C-2.5 Special Provisions

C-2.5.1 Concrete Structures

C-2.5.2 Steel Structures

The use of weathering steel is preferred because it requires less maintenance than painted steel. However, the use of weathering steel requires that special details be taken into consideration during desjgn, in order to prevent concrete and other surfaces from becoming stained by the weathering action of the steel after construction. An example of this is to provide a lip on the concrete abutments and piers to catch rainwater runoff and to provide provisions for drainage of this water to an approved discharge point. Some applications may consider the painting of the ends of girders on weathering steel. In other cases, if the brown weathering patina is considered undesirable aesthetically, then the structure may have only the outside visible edges of the structure painted.

C-2.5.3 Ballast Deck Bridge Structures

C-2.5.4 Special Provisions for Railroad Electrification

C-2.6 Substructure

C-2.6.1 General Layout

C-2.6.2 Foundation Types

It is important that the foundation designer be familiar with the operating requirements of the railroad. There is only a limited amount of construction periods available during operations thus the foundation recommendations shall consider working around and adjacent to live railroad tracks. The geotechnical specialists will be required to correlate their reports with the final design and vice versa. The recommendation in the geotechnical report shall be consistent with the selected foundation type. As an example, due consideration shall be given in the case of pile driving to evaluate the effects of vibration on adjacent lifeline and other pipelines, and adjacent structures that may or may not be owned by the PCJPB.

C-2.7 Construction Specifications

C-3. Design Guideline for GRADE SEPARATIONS OVER RAILROAD

C-4. Design Guideline for SEISMIC DESIGN

C-5. Design Guideline for PEDESTRIAN UNDERPASS

C-6. Design Guideline for RETAINING WALLS

C-6.1 General Requirements

C-6.1.1 Design Considerations

C-6.2 Selection of Types of Retaining Walls

C-6.2.1 Factors for Selecting Retaining Wall Types

C-6.3 Structural Design Requirements

C-6.3.1 Wall Design Heights

C-6.3.2 Loads

(b) Applied Loading Exclusive of Earth Pressure

The surcharge loading generally will consist of the Cooper E-80 loading conditions (or the E-XX chosen for the design). It is applied vertically at the bottom of tie elevation. It can be determined by taking the maximum axle loads and axle spacing and determining the vertical load to be applied in the Boussinesq strip load equation. Typically this is four 80-kip axles spaced at 5 feet off center, which would be 4 by 80k/4 by 5 feet (spacing) by 9 feet (tie length) or 1.778 kips per square foot (ksf). It is appropriate to use the 1.778 ksf load in the Boussinesq' strip load equation because the axle loading can occur anywhere along the track (strip) as long as the walls are parallel to the track. It is also appropriate to use the 1.778 ksf surcharge when designing walls perpendicular to the track (e.g., abutments).

C-6.3.3 Retaining Wall Stability

C-6.3.4 Abutment Stability

C-6.4 Clearances

C-6.5 Special Provisions10

C-6.5.1 Railroad Electrification

C-6.5.2 Utilities

C-6.5.3 Drainage

C-6.5.4 Mechanical Stabilized Embankments (MSE)

C-6.5.5 Crib Walls

C-6.5.6 Bulkhead Systems

C-6.5.7 Tieback Anchored Piles

C-7. Design Guideline for STRUCTURE DESIGN SUBMITTAL PROCEDURES

C-7.1 General

C-7.2 Design Submittals, Master List

C-7.2.1 Conceptual Design Level

C-7.2.2 Preliminary Design

C-7.2.3 In-Progress Design Level

C-7.2.4 Pre-Final Design Level

C-7.2.5 Final Design Level

C-7.2.6 Bid Documents

C-7.2.7 Bid Period Addendum Documents

C-7.2.8 Conformed Contract Documents for Construction

C-7.2.9 As-Built Documents

C-7.3 Submittal Quantities

C-7.3.1 Conceptual Design Level

C-7.3.2 Preliminary Design Level

C-7.3.3 In-Progress Design Level

C-7.3.4 Pre-Final Design Level

C-7.3.5 Final Design Level

C-7.3.6 Bid Documents

C-7.4 Construction Requirements for Structures

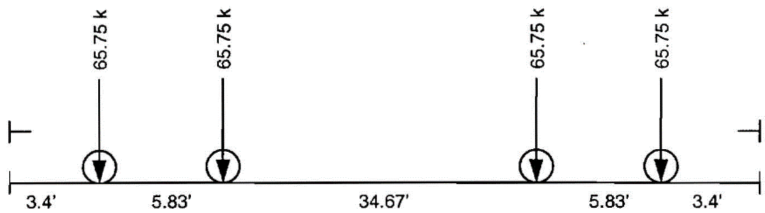
C-7.4.1 Construction Management

C-7.4.2 Construction Submittals

C-7.4.3 As-Built Submittals

APPENDIX A
EQUIVALENT E80 LOAD FOR EQUIPMENT

LOAD AND SPACING CHART FOR A TYPICAL 100-TON FREIGHT CAR

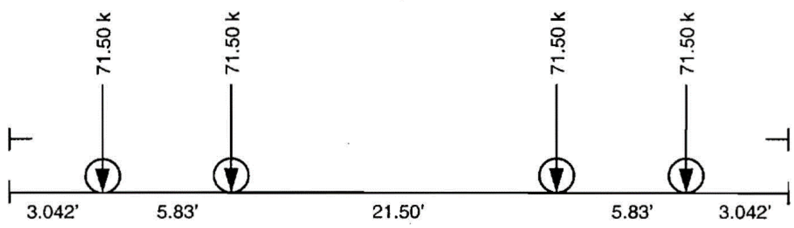


Coupler to Coupler Length: 53.13 feet

SPAN	BENDING		END SHEAR		FLOOR BEAM REACTION	
FT	FT-KIPS	E	KIPS	E	KIPS	E
8	131.5	65.8	83.6	60.8	93.4	53.4
10	165.0	58.7	93.2	62.1	114.2	57.1
12	226.1	56.5	99.6	56.9	128.0	54.9
13	257.2	54.1	103.9	56.3	135.2	54.9
14	288.5	52.5	110.6	57.3	144.4	55.4
15	324.8	52.0	116.3	58.2	152.3	55.7
16	374.1	53.4	121.4	57.1	159.2	56.0
18	472.7	55.6	129.8	55.6	170.7	56.3
20	571.3	55.4	141.6	56.7	180.0	54.9
25	843.7	55.3	165.9	58.7	196.6	52.0
30	1,167.4	56.9	182.1	57.8	207.6	48.1
35	1,492.5	57.1	193.6	56.0	215.5	44.2
40	1,818.6	55.5	202.3	53.7	221.5	40.9
45	2,145.2	53.6	209.1	51.2	232.7	39.1
50	2,472.3	52.0	214.5	49.2	250.7	38.8
60	3,127.2	48.2	231.2	47.2	301.9	39.3
70	3,782.9	44.3	258.5	46.8	366.0	41.2
80	4,442.9	41.0	290.6	46.8	418.9	41.9
90	5,354.5	40.0	316.8	46.2	460.0	41.9
100	6,558.6	40.6	337.7	45.0	497.5	41.7
120	9,152.2	39.7	381.6	43.9	602.5	44.0
140	12,827.7	41.2	437.6	44.5	704.3	45.5
160	16,756.4	41.9	481.7	44.0	792.3	45.9
180	20,701.4	41.9	532.3	44.2	900.2	47.0
200	24,873.6	41.7	584.3	44.6	994.3	47.3

MOMENT AND SHEAR TABLE FOR 100-TON CAR (263,000 LBS.) (STRING OF 10 CARS)

LOAD AND SPACING CHART FOR A STANDARD AAR 286,000-LB FREIGHT CAR

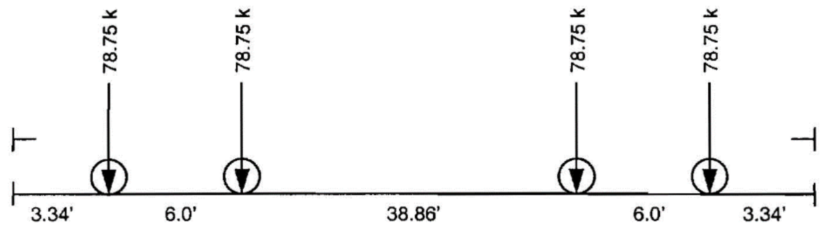


Coupler to Coupler Length: 41.88 feet

SPAN	BENDING		END SHEAR		FLOOR BEAM REACTION	
FT	FT-KIPS	E	KIPS	E	KIPS	E
8	143.0	71.5	90.9	66.1	108.0	61.7
10	179.5	63.8	101.3	67.5	129.3	64.7
12	245.9	61.5	108.8	62.2	144.0	61.7
13	279.7	58.9	116.9	63.3	154.9	62.9
14	324.9	59.1	123.9	64.2	164.3	63.0
15	378.5	60.6	129.9	65.0	172.4	63.1
16	432.1	61 .7	135.2	63.6	179.5	63.1
18	539.3	63.5	145.0	62.2	191.3	63.1
20	646.6	62.7	159.1	63.7	200.8	61.3
25	962.1	63.1	184.5	65.3	217.9	57.6
30	1,315.2	64.1	201.4	63.9	235.6	54.6
35	1,669.6	63.8	213.5	61.7	260.0	53.3
40	2,024.7	61.8	223.9	59.4	292.8	54.1
45	2,380.4	59.5	238.8	58.5	331.7	55.7
50	2,736.4	57.6	257.7	59.1	376.6	58.3
60	3,547.3	54.6	300.4	61.3	455.5	59.3
70	4,634.2	54.3	339.2	61.4	516.5	58.1
80	6,034.7	55.7	369.6	59.5	584.2	58.5
90	7,532.7	56.2	404.6	59.0	663.1	60.3
100	9,419.3	58.3	445.2	59.4	739.5	62.0
120	13,680.9	59.3	515.3	59.2	875.6	64.0
140	18,086.7	58.1	590.5	60.0	1,027.8	66.4
160	23,366.5	58.5	661.0	60.4	1,167.0	67.5
180	29,841.2	60.3	736.0	61.2	1,317.4	68.9
200	36,976.8	62.0	806.7	61.6	1,458.4	69.4

MOMENT AND SHEAR TABLE FOR 110-TON CAR (286,000 LBS.) (STRING OF 10 CARS)

LOAD AND SPACING CHART FOR A TYPICAL 125-TON FREIGHT CAR

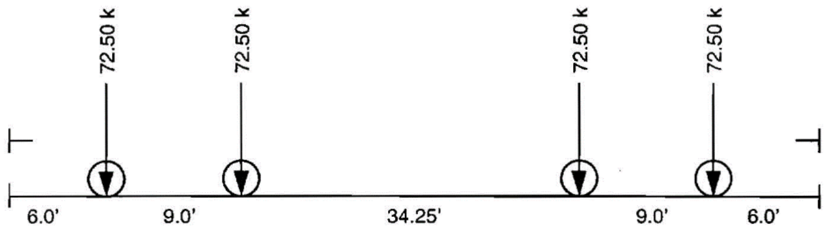


Coupler to Coupler Length: 57.54 feet

SPAN	BENDING		END SHEAR		FLOOR BEAM REACTION	
FT	FT-KIPS	E	KIPS	E	KIPS	E
8	157.5	78.8	98.4	71.6	111.4	3.7
10	196.9	70.0	110.3	73.5	136.4	68.2
12	265.8	66.4	118.1	67.5	153.0	65.6
13	302.9	63.8	123.1	66.7	161.4	65.6
14	340.3	61.9	131.2	68.0	172.4	66.1
15	386.9	61.9	138.2	69.1	181.9	66.5
16	445.9	63.7	144.3	67.9	190.2	66.9
18	564.0	66.4	154.5	66.2	204.1	67.3
20	682.1	66.1	167.9	67.2	215.1	65.6
25	1,005.3	65.9	197.3	69.8	235.1	62.2
30	1,393.2	67.9	216.9	68.8	248.4	57.6
35	1,782.8	68.2	230.9	66.8	257.9	52.9
40	2,173.4	66.3	241.4	64.0	265.1	49.0
45	2,564.7	64.1	249.6	61.1	270.9	45.5
50	2,956.5	62.2	256.2	58.7	283.2	43.8
60	3,741.1	57.6	269.2	54.9	331.1	43.1
70	4,526.5	53.0	294.3	53.3	398.9	44.9
80	5,312.4	49.1	329.9	53.1	466.9	46.7
90	6,137.9	45.8	363.2	52.9	520.0	47.3
100	7,298.6	45.2	389.9	52.0	562.5	47.2
120	10,333.0	44.8	433.2	49.8	659.1	48.2
140	13,971.6	44.9	493.6	50.2	783.9	50.7
160	18,676.3	46.7	550.0	50.2	882.9	51.1
180	23,401.3	47.3	597.7	49.7	987.9	51.6
200	28,126.4	47.2	657.4	50.2	1,107.5	52.7

MOMENT AND SHEAR TABLE FOR 125-TON CAR (315,000 LBS.) (STRING OF 10 CARS)

LOAD AND SPACING CHART FOR A PCJPB LOCOMOTIVE

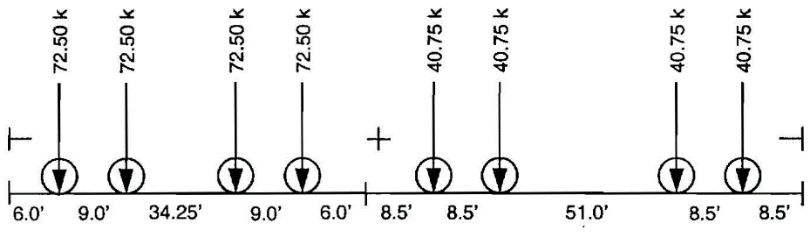


Coupler to Coupler Length: 64.25 feet

SPAN	BENDING		END SHEAR		FLOOR BEAM REACTION	
FT	FT-KIPS	E	KIPS	E	KIPS	E
8	145.0	72.5	72.5	52.7	72.5	41.4
10	181.3	64.4	79.8	53.2	79.8	39.9
12	217.5	54.4	90.6	51.8	90.6	38.8
13	235.6	49.6	94.8	51.4	100.4	40.8
14	253.8	46.1	98.4	51.0	108.8	41.7
15	271.9	43.5	101.5	50.8	116.0	42.4
16	299.6	42.8	104.2	49.0	122.3	43.0
18	367.0	43.2	108.8	46.6	132.9	43.8
20	435.5	42.2	112.4	45.0	141.4	43.1
25	609.4	40.0	130.5	46.2	168.2	44.5
30	871.8	42.5	145.0	46.0	188.5	43.7
35	1,143.4	43.7	165.7	47.9	203.0	41.6
40	1,442.8	44.0	181.3	48.1	213.9	39.5
45	1,798.0	44.9	193.3	47.3	225.2	37.8
50	2,154.7	45.3	203.0	46.5	238.9	37.0
60	2,871.0	44.2	217.5	44.4	274.6	35.7
70	3,589.8	42.0	233.8	42.3	319.8	36.0
80	4,310.1	39.8	256.0	41.2	370.9	37.1
90	5,080.9	37.9	279.7	40.8	422.1	38.4
100	6,045.1	37.4	306.7	40.9	466.9	39.2
120	8,438.1	36.6	352.2	40.5	544.1	39.8
140	11,287.5	36.3	392.0	39.9	634.9	41.0
160	14,835.3	37.1	439.1	40.1	732.3	42.4
180	18,995.0	38.4	487.0	40.5	815.3	42.6
200	23,345.0	39.2	527.9	40.3	904.2	43.0

MOMENT AND SHEAR TABLE FOR 4 PCJPB LOCOMOTIVES (STRING OF 4 – F40PH-2C (BLC))

LOAD AND SPACING CHART FOR A PCJPB LOCOMOTIVE + PASSENGER CAR



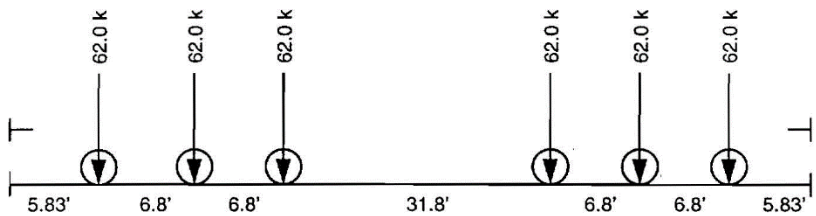
Locomotive Coupler to Coupler Length: 64.25 ft
Locomotive Weight :290 k

Passenger Car Coupler to Coupler Length: 85 ft
Passenger Car Weight: 127 k
240 Passengers at 150 lbs: 36 k
163 k

SPAN FT	BENDING FT-KIPS	BENDING E	END SHEAR KIPS	END SHEAR E	FLOOR BEAM REACTION KIPS	FLOOR BEAM REACTION E
8	145.0	72.5	72.5	52.7	72.5	41.4
10	181.3	64.4	79.8	53.2	79.8	39.9
12	217.5	54.4	90.6	51.8	90.6	38.8
13	235.6	49.6	94.8	51.4	94.8	38.5
14	253.8	46.1	98.4	51.0	98.4	37.7
15	271.9	43.5	101.5	50.8	102.9	37.6
16	299.6	42.8	104.2	49.0	108.0	38.0
18	367.0	43.2	108.8	46.6	116.7	38.5
20	435.5	42.2	112.4	45.0	123.6	37.7
25	609.4	40.0	121.3	42.9	139.3	36.8
30	785.7	38.3	132.1	41.9	153.8	35.7
35	1,003.8	38.4	143.2	41.4	164.2	33.7
40	1,235.9	37.7	153.6	40.8	172.0	31.8
45	1,476.6	36.9	161.7	39.6	181.9	30.6
50	1,757.9	37.0	168.2	38.6	200.8	31.1
60	2,321.3	35.7	177.9	36.3	229.3	29.8
70	2,885.5	33.8	184.9	33.5	256.0	28.8
80	3,450.3	31.9	204.5	32.9	280.6	28.1
90	4,147.0	31.0	223.0	32.5	303.0	27.6
100	5,070.3	31.4	237.9	31.7	325.5	27.3
120	7,008.0	30.4	260.1	29.9	366.6	26.8
140	9,210.0	29.6	287.4	29.2	410.4	26.5
160	11,225.2	28.1	310.2	28.3	451.6	26.1
180	13,632.8	27.6	335.7	27.9	491.5	25.7
200	16,274.8	27.3	362.6	27.7	530.1	25.2

MOMENT AND SHEAR TABLE FOR PCJPB 1 LOCOMOTIVE + 5 PASSENGER CARS
(1 – F40PH-2C LOCOMOTIVE (BLC) WITH 5-GALLERY CARS)

LOAD AND SPACING CHART FOR A SD-45 FREIGHT LOCOMOTIVE

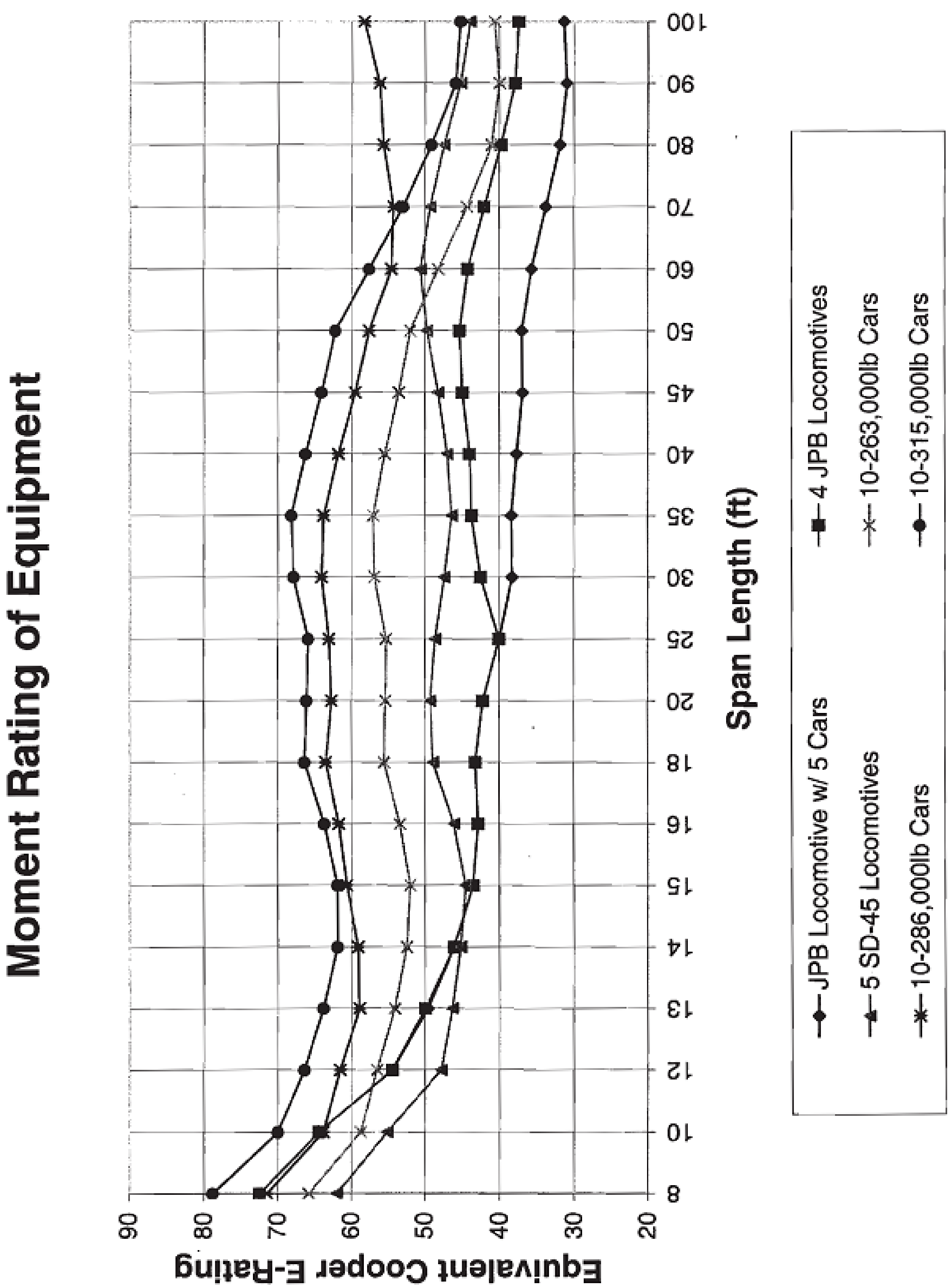


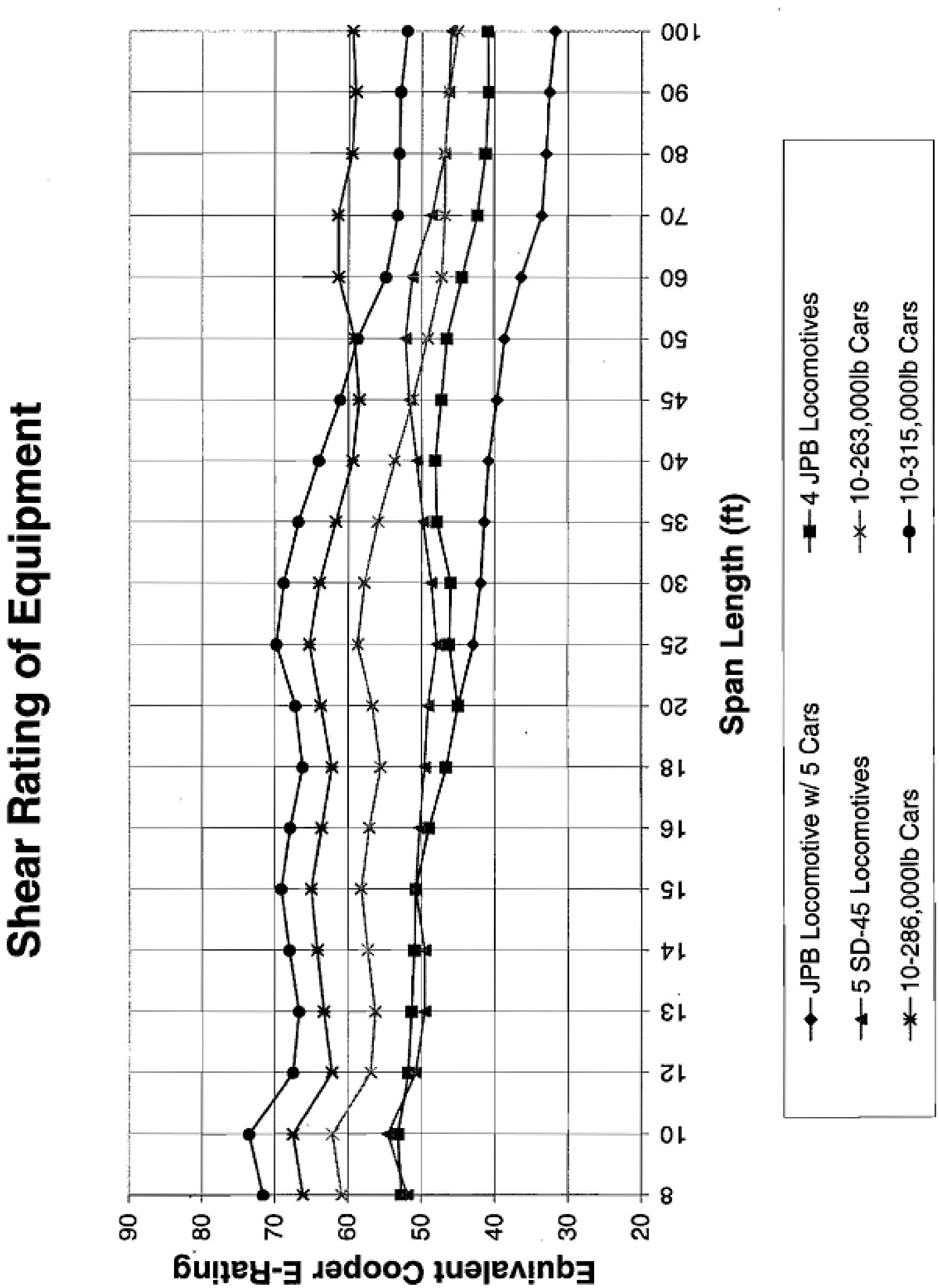
Coupler to Coupler Length: 70.66 feet

SPAN	BENDING		END SHEAR		FLOOR BEAM REACTION	
FT	FT-KIPS	E	KIPS	E	KIPS	E
8	124.0	62.0	71.3	51.9	80.6	46.1
10	155.0	55.1	81.8	54.6	101.7	50.8
12	191.1	47.8	88.9	50.8	115.7	49.6
13	219.8	46.3	91.6	49.6	121.1	49.2
14	248.8	45.2	95.7	49.6	125.8	48.2
15	278.1	44.5	101.7	50.8	129.8	47.5
16	322.4	46.1	106.9	50.3	133.3	46.9
18	415.4	48.9	115.7	49.6	139.2	45.9
20	508.4	49.3	122.8	49.1	153.4	46.8
25	740.9	48.6	135.4	47.9	184.7	48.8
30	973.4	47.4	153.6	48.7	215.4	49.9
35	1,213.9	46.4	172.3	49.8	237.8	48.7
40	1,541.2	47.0	191.3	50.7	254.5	47.0
45	1,927.9	48.2	211.4	51.7	267.6	44.9
50	2,364.0	49.7	227.4	52.2	283.7	43.9
60	3,283.5	50.6	251.5	51.3	324.9	42.3
70	4,206.0	49.3	268.7	48.6	369.1	41.5
80	5,130.3	47.4	290.9	46.8	426.6	42.7
90	6,055.9	45.2	317.6	46.3	483.8	44.0
100	7,101.2	43.9	344.2	45.9	543.3	45.6
120	9,826.9	42.6	404.5	46.5	640.8	46.8
140	13,233.1	42.5	453.0	46.0	737.6	47.7
160	17,064.3	42.7	503.2	46.0	847.5	49.1
180	21,768.8	44.0	557.5	46.3	957.7	50.1
200	27,165.3	45.6	613.3	46.8	1,056.1	50.2

MOMENT AND SHEAR TABLE FOR A SD-45 FREIGHT LOCOMOTIVE
(STRING OF 5 LOCOMOTIVES)

APPENDIX B
MOMENT AND SHEAR RATING OF EQUIPMENT


Moment Rating of Equipment
Equivalent Cooper E-Rating
Span Length (ft)
JPB Locomotive w/ 5 Cars
4 JPB Locomotives
5 SD-45 Locomotives
10-263,000lb Cars
10-286,000lb Cars
10-315,000lb Cars


Shear Rating of Equipment
Equivalent Cooper E-Rating
Span Length (ft)
JPB Locomotive w/ 5 Cars
4 JPB Locomotives
5 SD-45 Locomotives
10-263,000lb Cars
10-286,000lb Cars
10-315,000lb Cars