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CHAPTER 10 - STRUCTURAL DESIGN

10.1 GENERAL

The purpose of this chapter is to provide guidance and techniques for designing bridges, retaining walls, tunnels, large span culverts and other structural items. The goal of a structural design is to produce a structure that (1) serves the purpose for which it is intended; (2) is capable of co-existing within its immediate environment without causing adverse impacts (either visual or physical); and (3) is economical from both a maintenance and construction point of view. Structural design requires a solid understanding of the techniques of structural analysis and the behavior of a structure under various loading conditions. Structural design also requires knowledge of concrete, steel, and timber material properties.

Awareness of factors related to other engineering fields such as hydraulics and soils is necessary to ensure that the structure functions without affecting or being affected by its environment in a detrimental way. Finally, the importance of aesthetic appeal must be recognized to make the structure an extension of nature rather than an intrusion on nature.

The Federal Lands Highway Division (FLHD) Offices employ a staff of professional structural/bridge engineers who develop plans and specifications for projects and occasionally oversee the actual construction.

Since structural elements do not normally comprise the entire highway project, the structural engineer will generally function as part of a design team.

The roadway designer has the overall responsibility for seeing that all aspects of the project are addressed. However, the structural designer must obtain supporting data from the environmental, geotechnical, survey, and hydraulics staff and coordinate the structural design with these technical units. The structural engineer is responsible for the following:

- Developing bridge type, size and location (TS&L).
- Designing bridges, retaining walls, and other structures.
- Preparing complete PS&E's for structures.
- Providing technical assistance to construction staff.
- Checking contract shop drawings.
- Providing technical assistance to other agencies as requested.

A. Bridges. Bridges are the most common major structure encountered in highway engineering and the most varied in design. Bridges range from simple designs (such as a timber deck on stringers that are supported at each end) to very complex designs (such as segmental, cable-stayed, or suspension bridges). Span lengths can vary from 6 meters to hundreds of meters. Each bridge location is different and in most cases it is necessary to custom-fit a bridge structure into its surroundings. This generally precludes the use of wholly standardized plans and specifications in the design of bridges and requires that each bridge be handled individually.

Structural engineering work consists of designing new structures and repairing or rehabilitating existing ones. Bridges include both simple and continuous span structures constructed of reinforced concrete, prestressed concrete, steel, timber, or a combination of these materials. Span lengths generally range from 6 meters to approximately 60 meters. As a general rule, slab-type superstructures (either cast-in-place or precast, prestressed units) are economical for span lengths up to 15 meters. Superstructures consisting of a deck slab supported by stringers are commonly used for spans up to 30 meters. Structures with span lengths in excess of 30 meters require special consideration.

Bridge rehabilitation most often involves the repair of concrete decks which have been damaged by corrosion of the steel reinforcing in the deck. The type of repair needed depends on the level of concrete and steel deterioration. A deck that is severely deteriorated may have to be entirely replaced, whereas one that is moderately deteriorated could be made usable by removing and replacing all unsound materials. For decks in the initial stages of deterioration, one preventive solution may be to install a cathodic protection system to stop further corrosion.

B. Special Designs. The structural engineers may occasionally become involved with certain types of bridges or other structures which differ from those normally handled and would therefore be considered special designs. This category includes major bridges having exceptionally long spans and/or requiring unique design and construction techniques. Examples are cable-stayed bridges, segmental bridges, and long-span box girder bridges. Designing these types of structures often requires specific expertise. For this reason, the Washington Headquarters Bridge Division often reviews projects of this type and is available to provide assistance upon request.

In general, structures such as retaining walls, box culverts, and sign supports lend themselves to a standardized design. This enables the roadway designer to handle these types of structures with little or no assistance from the structural engineer. Occasionally, standard designs or plans are not entirely applicable to the conditions encountered, and a modified or custom design is necessary.

An example of a modified standard design would be a box culvert that is required to have dimensions larger than what are detailed in the standard plans. The structural engineer would then be responsible for developing plans and specifications for the structure. It is therefore important that the structural engineer understand the principles governing the design of these structures and also that the engineer recognize the factors which influence their design.

In addition to the structure itself, the structural engineer is sometimes called upon to design structural components for guardrails, sign supports, lighting supports, pedestrian screening, etc.

1. Retaining Walls. The retaining wall as a highway structure serves either to maintain the stability of a roadway embankment in fill areas or to prevent unstable material from sloughing off onto the roadway surface in cut areas.

The design of retaining walls is normally carried out by the roadway designer through the use of standard designs. However, this approach is not always practical. If wall height, foundation material, or the material being retained differs significantly from the design criteria on which the standardized designs are based, the structural engineer will custom design the installation.

2. Tunnels. Because of their high construction costs, highway tunnels have limited use and should only be considered when other more cost-effective alternatives are not practical. The successful design of a tunnel is dependent upon a comprehensive geologic study performed by qualified geotechnical engineers to determine the presence of faults, badly fractured rock, seams, water, etc. It is vital that the structural engineer work closely with the geotechnical engineers to determine requirements for lining, drainage, and methods of excavation.

3. Culverts. Culverts with clear spans greater than 3 meters are generally described as large culverts and are in most instances designed for a specific site condition by a structural engineer. While these structures are described as culverts, they are in most cases not used as drainage structures, but are used to pass farm livestock, farm machinery, industrial equipment, or people through an earth embankment. Typically, these large culverts are low profile steel arch superspans with spans from 6 to 12 meters, rigid frame reinforced concrete box structures with spans in the 4- to 5.5-meter range, and precast prestressed concrete low profile arch structures with spans in the 9- to 12-meter range.

10.2 GUIDANCE AND REFERENCES

The FLH program includes a wide variety of bridge types, site conditions, and design loadings. Accordingly, the bridge engineer relies on a wide variety of references for assistance.

A. Professional Assistance. The primary source of professional assistance is the FLHD bridge engineers and senior structural engineers within the design office. These individuals can provide not only technical guidance but also can explain the correlation between theory and specifications.

Additional professional assistance is available from the Bridge Division in the Federal Highway Administration, Office of Engineering, Washington, DC.

On FLH projects that become part of State highway systems upon completion of construction, State highway departments are also a source of excellent professional assistance.

As a matter of good office practice, all outside contacts should be informally discussed with the FLHD Bridge Engineer prior to making contact and the items discussed should be documented in the design notes or in the design files.

B. Design Specifications and Guidelines. The primary design specification for all highway bridges on public roads in the United States is the *Standard Specifications for Highway Bridges* published by American Association of State Highway Transportation Officials (AASHTO). It is also the primary design specification for all FLH bridges.

AASHTO specifications set forth *minimum* requirements that are consistent with current practice and certain modifications may be necessary to suit local conditions. AASHTO specifications apply to ordinary highway bridges, but supplemental specifications may be required for unusual types and for bridges with spans longer than 150 meters.

Interim specifications are published yearly by AASHTO and have the same status as standard specifications. Interim specifications are revisions that have been approved by at least a two-thirds majority of the members of the AASHTO Subcommittee on Bridges and Structures. FLHO policy is to apply Interim specifications to all design projects started after issuance of the specifications. Interim specifications shall not apply to projects retroactively.

The following AASHTO specifications including current revisions apply to all FLH bridge projects:

Standard Specifications for Highway Bridges. AASHTO. 15th ed. 1992 (with all current Interim Specifications).

Guide Specifications for Horizontally Curved Highway Bridges. AASHTO. 1993.

Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members. AASHTO. 1978 (with all current Interim Specifications).

Standard Specifications for Moveable Highway Bridges. AASHTO. 1988.

Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals. AASHTO. 1985.

Bridge Welding Code. ANSI/AASHTO/AWS D1.5-88. 1988.

Guide Specifications for Strength Design of Truss Bridges (Load Factor Design). AASHTO. 1986.

Guide Specifications for Design and Construction of Segmental Concrete Bridges. AASHTO. 1989.

The following specifications offer insight to and clarification of many of the AASHTO specifications:

Building Code Requirements for Reinforced Concrete and Commentary. ACI 318M-89. American Concrete Institute. 1992.

Ontario (Canada) Highway Bridge Design Code and Commentary. Ministry of Government Services, Toronto, Ontario. 1983.

AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings and AISC Code of Standard Practice. Current edition. (Found in Manual of Steel Construction, 9th ed., American Institute of Steel Construction.)

National Design Specification for Wood Construction and Design Values for Wood Construction. National Forest Products Association. 1991.

Design Standard Specifications for Structural Glued Laminated Timber. AITC 117-93 . American Institute of Timber Construction.

Structural Welding Code-Steel. American Welding Society. 1992.

Manual for Railway Engineering. 2 volumes. American Railway Engineering Association.

C. Design Examples. Previous projects are an excellent source of guidance.

Engineers with minimal experience should rely on the design notes and project plans of previous bridge projects. Care should always be exercised to select projects designed and checked by experienced structural engineers. Also, previous notes should not be followed in a cookbook manner, but rather, they should be used in conjunction with current AASHTO specifications.

Design engineers should always review new projects with the bridge engineer or with senior structural engineers before work is started. At this time, a similar example project to be used for guidance can be selected and discussed.

D. Technical References. State-of-the-art bridge design involves the practical application of the principles of many varied disciplines. The following references are listed to provide entry level structural engineers with theoretical background and assistance in practical bridge design. These references should not necessarily be considered FLHO policy. Experienced structural engineers may also find the listing useful for a personal library.

Structural Analysis

Moments, Shears, and Reactions for Continuous Highway Bridges. American Institute of Steel Construction. 1966.

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Handbook of Frame Constants. Portland Cement Association. 1958.

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Ketter, Robert L.; Lee, George C.; and Prawel, Sherwood P. *Structural Analysis and Design*. New York. McGraw-Hill Book Company. 1979.

Reinforced Concrete

ACI Manual of Concrete Practice. American Concrete Institute. 5 volumes. 1994.

Notes on Load Factor Design for Reinforced Concrete Bridge Structures. Portland Cement Association. 1974.
Analysis and Design of Reinforced Concrete Bridge Structures. American Concrete Institute. 1977.

Notes on ACI 318-833. Portland Cement Association. Current edition.

Ferguson, Phil M. *Reinforced Concrete Fundamentals, with Emphasis on Ultimate Strength*. 3rd ed. Salt Lake City. John Wiley and Sons.

CRSI Handbook. Concrete Reinforcing Steel Institute. 1992.

Degenkolb, Oris. *Concrete Box Girder Bridges.* Iowa State University and American Concrete Institute. 1977.

Heins, Conrad P. and Lawrie, Richard A. *Design of Modern Concrete Highway Bridges*. Salt Lake City. John Wiley and Sons. 1984.

Manual of Standard Practice. Concrete Reinforcing Steel Institute. 1990.

Reinforcing Bar Detailing. Concrete Reinforcing Steel Institute. 1990.

Hurd, M.K. *Formwork for Concrete*. 5th ed. American Concrete Institute. 1989.

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Merritt, Fredrick S. *Structural Steel Designer's Handbook*. New York. McGraw-Hill Book Company.

Highway Structures Design Handbook. American Institute of Steel Construction. 2 volumes. 1986.

Fischer, John W. *Bridge Fatigue Guide*. American Institute of Steel Construction. 1977.

Fischer, John W. *Fatigue and Fracture in Steel Bridges*. Salt Lake City. John Wiley and Sons. 1984.

Heins, Conrad P. and Firmage, D.A. *Design of Modern Steel Highway Bridges*. Salt Lake City. John Wiley and Sons. 1979.

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Composite Steel Plate Girder Bridge Superstructures. U.S. Steel Corporation. 1977.

Blodgett, Omar W. *Design of Welded Structures*. The Lincoln Electric Company.

Prestressed Concrete

Lin, T.Y. and Burns, Ned H. *Design of Prestressed Concrete Structures.* 3rd ed. Salt Lake City. John Wiley and Sons. 1981.

Post-Tensioning Manual. Post-Tensioning Institute. 1985 .

Post-Tensioned Box Girder Bridge Manual. Post-Tensioning Institute. 1978.

Precast Segmental Box Girder Bridge Manual. Post-Tensioning Institute and Prestressed Concrete Institute. 1978.

Libby, James R. *Modern Prestressed Concrete*. New York. D. Van Nostrand-Reinhold. 1977.

Prestress Manual. State of California Department of Transportation, Division of Construction. November 1978.

PCI Design Handbook, Precast and Prestressed Concrete. Prestressed Concrete Institute. 3rd ed. 1985.

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Timber Bridges: Design, Construction, Inspection and Maintenance. U.S. Department of Agriculture, U.S. Forest Service. June 1990.

Timber Construction Manual. American Institute of Timber Construction. Salt Lake City. John Wiley and Sons.

Weyerhaeuser Glulam Wood Bridge Systems. Weyerhaeuser Company. 1980.

Western Woods Use Book. Western Wood Products Association. 1978.

Timber Design and Construction Handbook. Timber Engineering Company. New York. McGraw-Hill Book Company. 1956. (Out of print.)

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Bowles, Joseph E. *Foundation Analysis and Design.* New York. McGraw-Hill Book Company. 1988.

Terzaghi, Karl and Peck, Ralph B. *Soil Mechanics in Engineering Practice*. Salt Lake City. John Wiley and Sons. 1967.

Cheney, Richard S. and Chassie, Ronald G. *Soils and Foundations Workshop Manual.* DOT, FHWA. Office of Highway Operations, National Highway Institute. 1982.

Manual on Design and Construction of Driven Pile Foundations. DOT, FHWA. 1985.

Soil Mechanics. Design Manual 7.1. Department of the Navy, Naval Facilities Engineering Command. 1982.

Foundations and Earth Structures. Design Manual 7.2. Department of the Navy, Naval Facilities Engineering Command. 1982.

Design of Piles and Drilled Shafts Under Lateral Load. DOT, FHWA. 1987.

Steel Sheet Piling Design Manual. U.S. Steel. (Updated and reprinted by DOT, FHWA, July 1984.)

Schnabel, Harry. *Tiebacks in Foundation Engineering and Construction.* New York. McGraw-Hill Book Company. 1982.

Woodward, Richard J.; Gardner, William S.; and Greer, David M. *Drilled Pier Foundations.* New York. McGraw-Hill Book Company. 1972.

Tiebacks. Report No. FHWA/RD-82/047. DOT, FHWA. Office of Research and Development. July 1982.

Permanent Ground Anchors. FHWA-DP-68-1. DOT, FHWA. Demonstration Projects Division. March 1984.

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Nathan M. Newmark and Emilio Rosenblueth. *Fundamentals of Earthquake Engineering*. Englewood Cliffs, NJ. Prentice-Hall. 1971.

Weigel, Robert L. *Earthquake Engineering*. Englewood Cliffs, NJ. Prentice-Hall. 1970.

Seismic Design of Highway Bridges - Workshop Manual. DOT, FHWA. Office of Research and Development, Implementation Division. January 1981.

Caltrans SEISMIC Bridge Design Specification and Commentary. California Department of Transportation, Office of Structure Design. 13th ed. 1983.

Miscellaneous Topics/Design Manuals

Bridge Design Practice - Load Factor. California Department of Transportation. October 1980.

California Falsework Manual. California Department of Transportation, Division of Structures. January 1988.

California Trenching and Shoring Manual. California Department of Transportation. May 1977.

Construction Handbook for Bridge Temporary Works. Report No. FHWA-RD-93-034. DOT, FHWA. Office of Research And Development. November 1993.

10.3 INVESTIGATION

In the development of structural design plans and specifications, the structural engineer will be confronted with data and comments obtained from several different types of investigations and reviews. This information may include bridge safety inspection structural condition data reports, bridge site survey information, and several levels of field review.

A. Bridge Site Plans. A bridge site plan is developed when a new or replacement bridge is required. The purpose of the site plan is to provide the structural engineer with a graphic representation of the topography at the site so the required type, size, and length of bridge can be determined for the site.

Bridge site topography can have a significant effect on the method of construction. The structural engineer must be aware of the possibilities and limitations that are presented by the existing conditions. Topographic maps assist the designer in determining quantities of excavation for estimating purposes.

The site plan shows the contours of the terrain as well as roads, streams, or other significant features in the immediate area of the proposed bridge. This data is collected by survey teams taking extensive topographic field measurements. The plans should be drawn using a scale appropriate for the total length of the proposed bridge. Contours are generally drawn at 0.5 or 1-meter intervals.

B. Hydraulic Analysis. In cases where a bridge crosses a river, stream, or flood plain, it is usually necessary to perform a hydraulic investigation and analysis. This is generally accomplished concurrently with the development of the site plan since hydraulic information is needed in deciding what type of structure is practical for the crossing. The structural engineer is interested in the high water elevation and flow velocity for flood conditions with a specified frequency of occurrence.

Typically, bridges are designed to handle a 50 year flood, which is a flood of such magnitude that it is expected to occur no more frequently than once in 50 years. For some large, high-cost structures, the design might be based on a 100 year flood to lessen the risk of flood damage. For detailed information with regard to the hydraulic design of bridges, see Chapter 7.

C. Geotechnical Investigation. Geotechnical investigations should be performed after the site plan has been developed and preliminary determinations have been made regarding the type and length of the proposed structure and the location of the foundations for the structure. The purpose of the investigation is to identify the composition of the underlying stratum, determine whether the preliminary location is acceptable as a foundation site, and determine what type of foundation design is most appropriate.

Most often the investigation consists of extracting and analyzing core samples of the substratum. Core drilling is normally performed to the depth necessary to reach solid rock. For small bridges on flat terrain, a single core is sometimes sufficient. For bridges longer than approximately 30 meters or bridges located on hilly terrain, a more comprehensive study is usually needed. It is desirable to obtain at least one core at each foundation site.

After analyzing the data, the geotechnical engineer should prepare a report containing recommendations for the type of foundation needed along with allowable bearing capacities and any other pertinent information. The structural engineer receives a copy of this report to assist in developing the final design of the foundations. For detailed information with regard to the geotechnical design of bridges, see Chapter 6.

D. Bridge Inspection Program. All bridges located on public roads are required by law to be inspected at regular intervals not to exceed 2 years. The inspections shall be in accordance with the National Bridge Inspection Standards and Guidelines as set forth in Title 23, Code of Federal Regulations, Part 650, Subpart C.

The Federal Lands Highway (FLH) Division Offices administer a bridge inspection program for the National Park Service and other Federal agencies. Bridge structures are reviewed for condition and structural adequacy.

Basic data that can be found in a typical inspection report includes the following:

- Photographs of the roadway and profile view of the bridge.

- A written description and photographs of deficiencies found during the inspection.

- Basic physical dimensions of the bridge.

- A structural load capacity rating, where applicable.

In many instances, data found in these reports is the basis for the development of preliminary bridge repair plans.

E. Deck Survey. A deck survey is performed to assess the structural condition of the bridge deck. The information is used by the structural engineer to determine if the deck can be repaired and the most suitable method of repair.

A deck survey may be composed of several types of investigations, which can be classified as either destructive or nondestructive. Half cell potential readings and delamination readings are non destructive since they provide information without actually disturbing the deck.

Destructive methods such as taking chloride samples and deck cores are generally used only when nondestructive methods yield data that indicates the potential or presence of severe internal deck deterioration. Chloride samples are taken to determine the level of chloride contamination in the deck.

Deck cores allow a visual inspection of the deck condition below the surface. Also, split-tensile tests can be performed on the cores to give an indication of the strength of the existing concrete.

F. RRR Bridges. A decision shall be made to retain or replace any bridge within the limits of an RRR project. See the applicable chapter of the AASHTO Green Book. When a bridge requires replacement, design the new bridge in accordance with AASHTO structural standards for bridges. Select widths consistent with current standards to which the highway may be upgraded in the near future. Review recent bridge inspection reports to determine if the bridge is structurally and functionally adequate.

When a bridge is to remain in place, make an evaluation to determine what treatment, if any, is required for operational and structural adequacy.

No work or only minor rehabilitation is required provided the following are:

- The bridge clear roadway width is equal to or greater than the minimum surfacing or approach traveled way widths.

- The bridge accident records indicate that accident problems do not exist and the approach is gradually narrowed to meet the bridge clear roadway width in advance of the bridge. When accident problems exist, make an analysis to determine the necessary corrective action such as providing improved transitions, rehabilitation, widening, or replacement.

- The bridge railings, including the approach rail, meet or are made to meet adequate strength and geometric standards. In all cases where a structure is to remain in-place, check the bridge rail for adequacy. When the existing bridge rail does not meet the current design standard, and it is not cost effective to bring it up to full standard, then treat it as an exception.

- A reasonable or adequate alternative route does not exist and the load carrying capacity is sufficient to carry school buses and vital service vehicles (M13.5 minimum design loading).

Consider major rehabilitation when:

- Deck replacement, to the extent practical, is designed in accordance with current AASHTO standards for highway bridges (M13.5 minimum design loading).

- Rehabilitation meets current AASHTO safety standards.

- Bridge railing is to be upgraded to current standards.

- The approach roadway width does not meet current AASHTO geometric standards and the bridge is to be widened to meet the geometric standards for the highway if it were reconstructed. The decision to rehabilitate or replace may be decided by established cost guidelines or may be subjective. However, when the total cost of rehabilitation is expected to exceed 50 percent of the cost of reconstructing the structure to current standards, consider replacing the structure.

Vertical clearances at existing underpass structures will require adjustment when the clearance after resurfacing work is less than the minimums required. Do not reduce surfacing depths or eliminate surfacing in the vicinity of the bridge to avoid pavement removal or structure modification. All signing and markings for bridges shall be in accordance with the MUTCD.

G. Field Reviews. Two levels of field reviews are generally required in the development of plans for bridge repair, replacement, or new construction. The first field review is designed to involve the responsible agencies in the design concepts and parameters that will be used in the development of plans and specifications for the given project. Basic information to be supplied by the structural engineer at this review is a proposed bridge type, size, and length (TS&L) drawing for replacement and new bridge projects. Drawings depicting proposed repair methods shall be provided for bridge repair projects.

The second level of field review, commonly known as a plan-in-hand review, should be performed when the bridge drawings are approximately 70 percent complete. The purpose of this review is to verify that all items covered in the drawings will be compatible with the existing field conditions and to confirm that all design, safety, and specific client agency needs are properly addressed in the final design documents.

10.4 DESIGN PROCESS.

The design process involves two stages. The initial or preliminary design effort establishes the proposed structure type and layout. The final design effort develops detailed contract plans to be used to construct the facility. Both of these stages require the skills of a structural engineer.

In the preliminary design process, a structure is selected which economically fulfills the structural, functional, aesthetic, and other relevant requirements of a given site.

The development of the preliminary plan requires the consideration of many different factors. The following are some of the more common of these factors:

- *Economic.* Initial costs; maintenance costs.

- *Site Requirements.* Topography; horizontal and vertical alignment; superelevation; deck geometrics; proposed or existing utilities.

- *Hydraulic.* Stream flow (Q_{50}, Q_{100}); risk assessment; passage of debris; scour; pier and bank protection; permit requirements; deck drainage; culverts (as alternatives).

- *Structural*. Span ratios; horizontal and vertical clearances; limitations on structural depth; future widening; slope treatment; foundation and groundwater conditions; anticipated settlement

- *Environmental.* Aesthetics and, compatibility with surroundings; similarity to adjacent structures; extent of exposure to the public.

- *Construction.* Access to site; time for construction; detours or stage construction; extent of falsework and falsework clearances; erection problems; ease of construction.

- *Safety*. Traffic convenience; density and speed of traffic; approach guardrail type and connection to structure; bridge rail type.

- *Other.* Recommendations resulting from interdisciplinary team studies or special requests by an owner.

In making the recommendation for type of structure, full consideration should be given to the above factors. Economy is generally the best justification for a selection. However, some of the above considerations may outweigh differences in cost. In the final analysis, the owner must be satisfied that the proper structure has been selected.

The final design process begins with the approval, by all interested parties, of the bridge TS&L drawing. Using the information shown on the drawing, and following the design specifications, the structural engineer makes a comprehensive analysis and design of the bridge. This design is then the basis for the preparation of detailed contract plans to be included in the complete project plans.

The final design of bridges requires meticulous attention to details and a high degree of responsibility. Irresponsible design can result in construction difficulties, reduced service life of the structure, and higher maintenance costs. In the extreme case, poor design can result in the collapse of the bridge either during construction or in service.

It is FLHO policy that a complete and independent check be made of all structural design work. (This means that one structural engineer designs the bridge and a second structural engineer performs an independent structural analysis of the bridge.)

The information that follows applies to both preliminary design and to final design.

A. General Features. The FLH program involves a wide variety of bridge types from single lane forest development roads to high volume urban arterials. The general features, including widths, clearances, railings, and approaches of these structures, are normally controlled by the roadway standards of the client agency. All necessary general features should be shown on the bridge TS&L and should be agreed upon before final design begins.

1. Bridge Widths and Clearances. Single lane bridges should be a minimum of 4.3 meters wide, face-of-rail to face-of-rail.

Multiple lane bridges should be as wide as the approach roadway plus the offset to the face of the approach guardrail.

Vertical clearances for interchange structures should meet AASHTO specifications or be consistent with other bridges on the route.

2. Bridge Railings and Approach Railings. Railings meeting both the geometric and structural requirements of AASHTO should be provided for all bridges. When client agencies request that railings be used that do not meet these requirements, take the following action:

- Document on the plans, under the specifications section of the general notes, the criteria that was used to design the railing.
- Document in the design file, the details of the design exceptions and who in the client agency was notified of these exceptions.

The use of approach railing on all bridges shall be encouraged. When approach railing is provided, it should be connected to the bridge railing system with connecting details that will develop the full strength of the approach railing.

All concrete parapet-type bridge railings should be detailed with joints as follows:

- At the point of maximum positive movement of all spans.
- At or near the centerline of all piers.
- In between the above locations such that the length of rail segments does not exceed 7.6 meters.
- At bridge expansion joints.

At these locations, joints should be detailed normal to the rails or radial on curved bridges. Joint filler material should be a minimum 12 millimeters thick. Reinforcement should not extend through the joint.

Joints for special design concrete railings should be located as necessary to control cracking due to flexure or temperature changes.

At the ends of the bridges, between the superstructure and substructure elements, railing joints should be compatible with deck joints, expansion assumptions, etc.

All steel bridge railing should have joints located as described above. Joints should be designed to allow movement that maintains the full strength of the railing.

3. Hydraulic Considerations. Most bridges are designed to pass, without damage, Q_{50} flows. The effects of Q_{100} flows should be evaluated. Normally, there are only minor differences in these two flows and most structures will pass both without damage. For details concerning other hydraulic considerations for scour, clearances, and slope protection, see Chapter 7.

B. Loads. Loads are fundamental to bridge design, having evolved through experience and study over many years. AASHTO has included design loads in bridge design specifications since the mid-1920's. The loads discussed in this manual reflect current bridge design criteria for ordinary highway bridges with spans less than 150 meters . These loads should be supplemented as necessary for unusual site conditions or traffic requirements.

1. Dead Loads. Structure dead loads are the loads imposed on a member by its own weight and the weight of all other structural elements that it supports. In addition to these loads, members must be designed to support the weight of superimposed dead loads such as wearing surfaces, rails and curbs, stay-in-place forms, and utility lines.

Designs should include provisions for an additional 1.2 kilonewtons per square meter of deck surface for future deck overlays.

Normally, the design engineer must make a preliminary estimate of the dead load of a member on which to base the initial design. The actual member weight must then be used to recalculate dead load effects, and the design must be checked. This iteration process converges quite quickly. The design notes should always contain the dead load effects of the final design.

It is possible to arrive at a final value of the dead load for one part of a bridge before proceeding with the design of a supporting part. For example, the deck is always designed before the girders. For this reason, all designs should proceed from the topmost members to the lowest members. (This is exactly the opposite order in which the bridge is constructed.)

2. Live Loads. A highway bridge should be designed to safely support, without permanent damage, all vehicles that might pass over it in its lifetime. In the United States, AASHTO specifies design live loads, and State laws specify the legal weights of motor vehicles. This combination of controls provides safety for our bridges. In addition, the owner of the structure may specify larger design live loads and thus may issue permits for heavier traffic vehicles.

a. M and MS Loads. The four classes of AASHTO loadings known as M13.5, M18, MS13.5, and MS18 were adopted in 1944 (previously referred to as H15, H21, HS15, and HS20, respectively). See *AASHTO Standard Specifications for Highway Bridges,* Article 3.7 Highway Loads. The vehicles are hypothetical and were not selected to resemble any particular existing design. However, any actual vehicle that would be permitted to cross a bridge should not produce stresses greater than those produced by the hypothetical vehicle.

MS18 is by far the most common live loading used today and most projects use this loading. Some NPS bridges are designed for M13.5 or MS13.5 live loading. A few states have adopted MS22.5 loading, which is the same axle spacing as MS18 loading with axle loads increased by 25 percent.

M and MS loadings in AASHTO are presented as both truck loads and lane loads. The MS truck loadings show a variable spacing of the two rear axles of 4.3 to 9.1 meters. The correct spacing is the length that produces the maximum effect.

Only one truck is applied, per lane, on the entire bridge at one time. The lane loadings are simplified loadings which approximate a train of trucks. Lane loadings include a concentrated load that is different for moment than for shear. Only one concentrated load is used in a simple span or for a positive moment in a continuous span. Two concentrated loads are used for negative moment in a continuous span. The uniform load portion may be divided into segments on a continuous span to produce the maximum effect.

Appendix A in the *AASHTO Standard Specifications for Highway Bridges* may be used as a guide to determine if lane or truck loading is the controlling factor.

b. Special Live Loads. Projects sometimes include bridges designed for special live loads. These may be logging vehicles, military transport vehicles, or heavy construction equipment.

Unless otherwise noted, these should be considered overload vehicles, and appropriate overstress or load factors should be used in design. (Details on these procedures are included in appropriate subsections of this manual.) It should be noted that most overload vehicles require careful analysis of the *lateral* distribution of the loads also.

c. Pedestrian Loads. Pedestrian bridges should be designed for a live load of at least 4.1 kilonewtons per square meter (4.1 kilopascals) of walkway. This load may be applied continuously or discontinuously over the length or width of the structure in order to produce the maximum stress in the member under consideration.

Pedestrian loads on sidewalks attached to highway bridges vary depending on the span and are detailed in the *AASHTO Standard Specifications for Highway Bridges*.

d. Impact. A vehicle moving across a bridge at a normal speed produces greater stress than the same vehicle in a static position on the bridge. This dynamic effect is known as impact. From the theory of dynamics, a load applied instantly to a beam causes stresses of twice the magnitude obtained when the same load is static on the beam.

In bridges, the loading is applied over a short period of time, but not instantly. Hence, impacts are less than 100 percent; specifically, they are less than 30 percent.

Impact (I) is determined by the formula:

$$I = \frac{15.24}{L + 38}$$

where:

L = Length of span of loaded portion of bridge, in meters.

Examples of "L" are shown in Figure 10-1.

e. Longitudinal Force. AASHTO specifications provide for longitudinal force for the traction and braking effects of vehicular traffic headed in the same direction.

This force, when combined with other forces, may affect the design of bents. Occasionally, in framed structures where the bents are very stiff, longitudinal force may affect superstructure design.

The application of the longitudinal force for framed bridges should be applied 1.8 meters above the deck. This does not change the girder moments significantly. It is important in the design of bearings and substructure. For bridges other than framed structures, the force should be applied as a shear force at the bearings.

Longitudinal force due to friction at expansion bearings or shear resistance of elastomeric bearings should also be considered. (More information is included in the section on bearings, later in this chapter.)

f. Centrifugal Force. Centrifugal forces are significant in the design of bridges having small curve radii or curved bridges supported by long columns. This force is applied 1.8 meters above the deck at the roadway centerline.

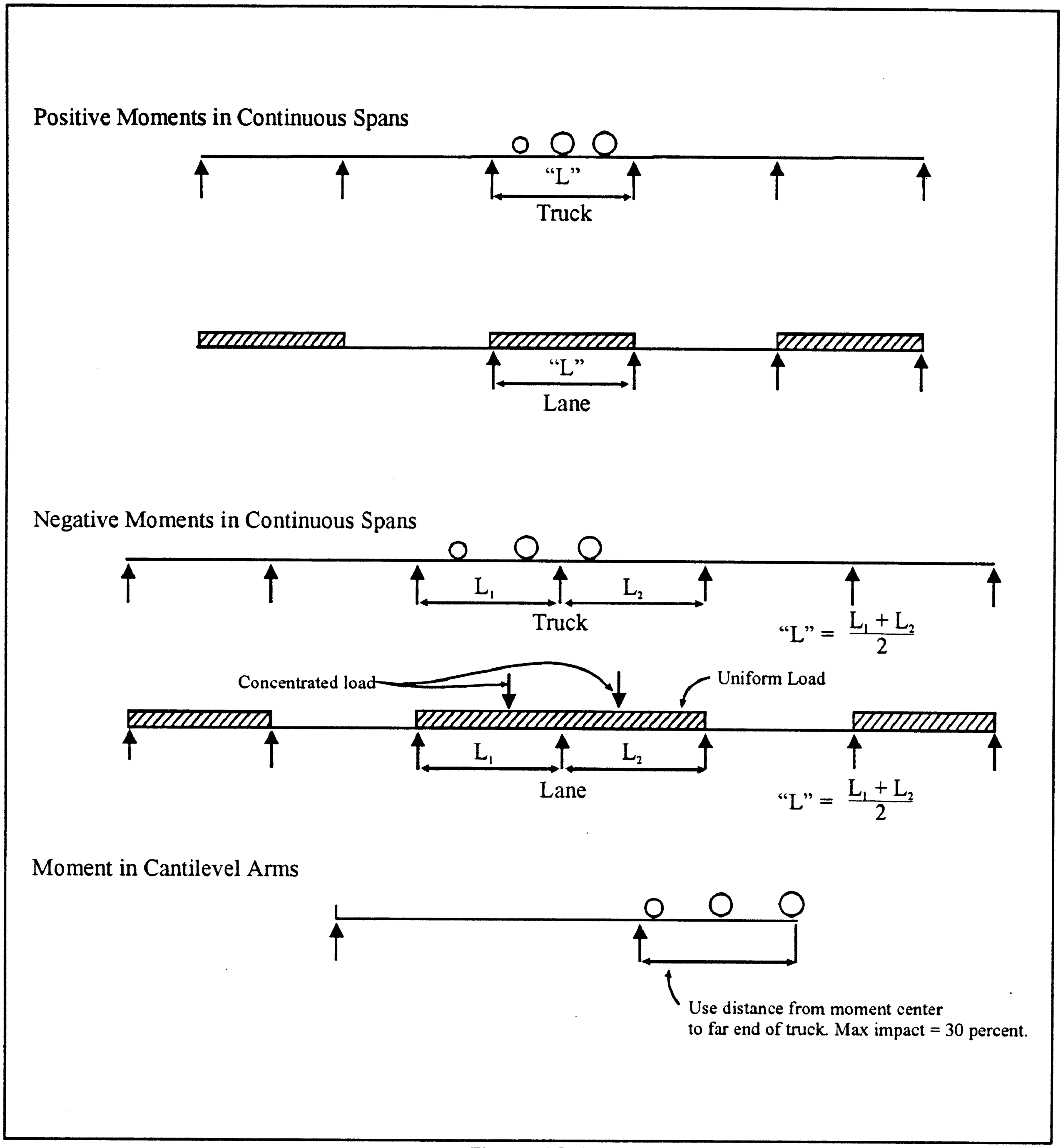

Figure 10-1
Impact Examples

a. Displacement of Supports. Possible displacement or settlement of supporting elements of a bridge should be considered in design. If settlements are unavoidable, and of possible large magnitude, provisions should be made in the design to periodically reset bearings to offset the effects of the settlement. Such structures require close coordination of the structural and geotechnical engineers.

b. Temperature. Forces caused by changes in temperature rarely control superstructure design. Forces can become significant and even unmanageable on frame structures with stiff columns. The design range of temperatures for concrete structures represents about one-half of the full air temperature range. This is due to the slow response of concrete structures to temperature changes.

c. Shrinkage. Shrinkage is the volume decrease that occurs when fresh concrete hardens and continues for a period of time thereafter. Shrinkage could be critical in arches (rib shrinkage produces rib and column moments) and in prestressed girders where shrinkage produces loss of stressing force. Shrinkage forces may be reduced by careful placement of mandatory construction joints.

4. Environmental Loads. Environmental loads include the effect of wind, stream and ice flow, buoyancy, and earth pressure.

a. Wind. The basic wind loads are from a high wind of 160 kilometers per hour and a moderate wind of 48 kilometers per hour. In general, the high wind is assumed to act on the bridge when live load is not present. Moderate wind acts on the bridge when live load is present for some load combinations.

Forces are applied in a variety of ways depending on whether one is designing a superstructure or substructure, and whether the structure is ordinary or unusual.

Horizontal wind loads on the superstructure are always based on the side view cross sectional area. They act both longitudinally and transversely. Loads on the substructure can be applied to side or transverse views, or skew angles in between.

The use of engineering judgment is sometimes necessary when considering wind velocities to be used in the design of a structure. Permanent terrain features or precise data from local weather service records may indicate that the basic 160 kilometers per hour design wind should be modified. When modified, the specified wind pressure is changed in the ratio of the square of the design wind velocity to the square of the base wind velocity. The revised design wind must be stated in the general notes of the bridge plans.

The wind effect on the bents and footings needs to be thoroughly investigated for high structures both laterally and longitudinally.

The limiting height of column where wind may control varies with the span lengths, physical makeup of the structure, and the magnitude of other lateral loads such as that due to earthquakes.

When applying lateral loads in continuous structures, consideration should be given to the rigidity of the deck and its ability to transfer wind loads to abutments which might be considerably stiffer than the bents. In these cases, the abutments shall be designed to support these lateral loads.

In addition to moderate wind pressure on the bridge structure when a live load is present, a moderate wind force is exerted on the live load itself. This force is expressed as a live load acting both transversely and longitudinally 1.8 meters above the roadway surface. This offset location is important when designing high piers.

b. Stream and Ice Flow. Stream flow rarely controls the design of bridge piers. Ice flow can result in very large stresses. The maximum ice loading often occurs simultaneously with high water conditions that take place in early spring. The thickness of the ice and the corresponding river level should be carefully determined before starting design. Very large ice loadings should be controlled with piers incorporating icebreaker noses and armor.

c. Buoyancy. Whenever a portion of a structure will be submerged, the effects of buoyancy should be considered in the design. In small structures, its effects are unimportant and no economical advantage can be realized in the footing design. In large structures, buoyancy effects should be taken into account in the design of footings, piles, and piers.

Buoyancy is always a consideration for footings below the water table where cofferdams are necessary. Uplift in piles is limited to an intermittent value of 40 percent of the allowable design load.

d. Earth Pressure. Abutments and retaining walls should be designed so that any hydrostatic pressure is minimized by providing adequate drainage for the backfill.

For level backfill, the minimum active earth pressure is usually taken as an equivalent fluid pressure of 5.7 kilonewtons per square meter (5.7 kilopascals) of height for abutments and retaining walls. This is based on an earth pressure coefficient (K_a) of 0.30 and unit weight (w) of compacted earth of 1900 kilograms per cubic meter. These numbers are used in the design of the following elements:

- Toe pressure or toe piles in retaining walls and abutments.
- Bending and shear in retaining walls and abutments.
- Sliding of spread footings or lateral loads in piles.

For the design of heel piles in retaining walls or abutments, checks should be made using an equivalent fluid pressure of 4.2 kilonewtons per square meter per meter (4.2 kilopascals per meter) of height. This corresponds to a K_a of 0.225.

A trapezoidal pressure distribution is used where the top of the wall is restrained. This provides a more realistic solution than the triangular pressure distribution that applies to typical retaining walls without restraint.

5. Earthquake Loads. Earthquakes and the response of structures to earthquakes, are dynamic events -- events that go into many cycles of shaking. An earthquake of magnitude 8+, such as that which occurred in San Francisco in 1906 and in Alaska in 1964, may have strong motions lasting for as long as 40 to 60 seconds. The San Fernando earthquake of magnitude 6.6 had about 12 seconds of strong motion. During this period of strong motion, a structure passes through many cycles of deflection in response to the motions applied at the base of the structure. The strains resulting from these deflections are the cause of the structural damage.

Structures that may be subjected to earthquake forces shall be designed to survive the strains resulting from the earthquake motion. Three factors that are considered when designing to resist earthquake motions are as follows:

- The proximity of the site to known active faults.

- The seismic response of the soil at the site.

- The dynamic response characteristics of the total structure.

It is FLHO policy to use the AASHTO *Guide Specifications for Seismic Design of Highway Bridges*, Chapter 3 instead of the AASHTO Equivalent Static Force Method since the guide specification contains significant improvements and helps the designer deal realistically with seismic bridge response and design methodology.

The guide specifications provide for different levels of analysis and design requirements for four seismic performance categories (A through D). Each bridge is assigned to one of these four categories depending on its potential seismic acceleration coefficient and an importance classification. No detailed analysis is required for any simple span bridge or any bridge in seismic performance category A; the only requirements pertain to connections and minimum support lengths.

The higher seismic performance categories, B, C, and D, require either a single mode or multimode spectral method of analysis depending on the bridge type and performance category. These methods are dynamic analyses which require the designer to learn both the basic principles in dynamics as well as proper structural modeling for computer analysis.

The single mode spectral method is used to calculate the seismic design forces of a bridge that can be characterized as having its major dynamic response in a single mode of vibration and is limited to the lower seismic structures. This method, although quite rigorous, reduces a complex dynamics analysis to the performance of just two static analyses and on certain, uncomplicated structures, can be done by hand.

The multimode spectral method is required for the higher seismic structures and can only practically be done by computer. It determines many different modes of vibration from a three-dimensional mathematical model of the structure along with response spectrum loadings to produce multimodal contributions to the overall seismic response of the structure.

A computer program currently in use, SEISAB (Seismic Analysis of Bridges), was specifically developed to help bridge designers conduct the guide specification seismic analyses of most conventional bridges with minimal input.

Since the guide specification contains new analysis and design methodologies, it is recommended that new users thoroughly study the guide specification commentary as well as the references listed under Section 10.2.

6. Combination Loads. AASHTO specified ten groups to represent various combinations of the previously discussed loads to which a structure may be subjected. Each component of the structure or the foundation on which it rests, is required to be designed for the most critical group. Allowance for the probability, frequency, and structural effect of these load combinations is made by an allowable overstress (percentage of basic unit stress) for service load design, and the load factors for load factor design. For example, only 30 percent of the full 160 kilometers per hour wind force is applied simultaneously with critical truck or lane live loading.

Group I is the everyday set of loads a structure is expected to resist that consists of dead load, design live load, centrifugal force, earth pressure, buoyancy, and stream flow. Under the Load Factor Design, a 1.67 factor is required for the design live load to ensure adequate strength for overloaded (allowed by permit) vehicles.

Group IA is required to ensure overload capacity for structures designed for live loads less than M18.

Group IB is included to ensure that structures will have adequate resistance to allow passage of normally allowed overload vehicles on a permit basis by the structure owner or regulatory agency. This provides that all structures on a given highway route are capable of meeting the owner's permit policy for transportation of infrequent extralegal overloads. It is FLHO policy to use the owner's permit loading or loading combinations for this group. (For example, Caltrans uses a family of overload vehicles called P-loads which are a set of five trucks, each composed of a steering axle and from two to six pairs of tandem axles at 5.5-meter centers. Each axle weighs approximately 1.5 times legal loads.)

Normally, the controlling loads for superstructure components are dead load and live load plus impact; however, the designer should always verify this. For example, a typical steel plate girder section is normally sized for dead load and live load plus impact. A check must be made for wind and wind on live load under groups II and III for flange stresses when wind bracing is omitted.

For substructure design, normally two groupings of the loads are required:

- A factored set for the load factor design of reinforced concrete columns.

- An unfactored set for the service load design procedure for sizing spread and pile supported footings as well as other foundation units.

Live load distribution to substructure units is done with whole truck or lane loadings rather than girder distribution, since girder distribution could accumulate more wheel lines than would physically fit on the bridge.

The AASHTO list of loads and groupings of loads is not meant to be all inclusive. For example, prestress frame shortening, snow and avalanche pressures, and construction loadings must be calculated and combined with other loads appropriately. An example of a construction loading to be checked during design is the buckling resistance of the composite compression flange of a plate girder during construction of the deck.

When checking foundation stability safety factors against overturning, sliding, etc.) neither load factors nor allowable overstresses should be used.

C. Decks, Rails, Deck Joints, and Drains. The roadway surface of bridges that support and contain vehicular traffic consists of the deck, rails, deck joints, and drains. This surface must not only provide a good riding surface but must also provide durability against abrasive deterioration and repetitive cycles of loading in flexure and shear.

1. Deck Design. Transversely reinforced concrete slabs are the most commonly used bridge deck and are a significant portion of bridge design in terms of dollar investment.

These slabs also make-up the one portion of the bridge that has the most common and expensive maintenance problems. Heavy wheel loads, excessive use of deicing salts, studded tires, and poor construction control are contributing factors to structure damage.

Edge support for transversely reinforced slabs is normally provided by cast-in-place end diaphragms. These diaphragms are often placed only between girders. Caution should be exercised to provide an edge support on slab overhangs where a substantial length of overhang might exist and where moments due to wheel loads might be a major portion of the total moment requirement. Cast-in-place decks on structural steel superstructures is another place where edge support might not naturally be provided. Edge support should be designed for each condition to be capable of carrying a wheel load.

Placement of transverse slab reinforcing on skewed bridges is a subject of some debate. A reasonable rule used by many designers, however, places the reinforcement on the skew for up to 20°, and for 20° or greater, places the reinforcing normal to the roadway with variable length bars at the skewed ends. For reinforcement placed on the skew, the span should be increased to the skewed length and the area of reinforcement increased for the spacing normal to the skew.

The AASHTO specifications require a 50-millimeter cover over the top reinforcing steel and a 25-millimeter cover over the bottom reinforcement in deck slabs. This means that the effective depth for a negative moment is less than that for a positive moment, and because transverse slab spans are designed for the same moment at midspan and at the support, the negative moment top reinforcement is more critical. Therefore, it is common practice to design the top reinforcement and to make the bottom reinforcement the same to avoid confusion during construction.

FHWA Federal-aid Policy Guide 23 CFR 650F, recommends increasing the cover over the top reinforcing steel to 65 millimeters. The purpose is to ensure that a minimum of 50 millimeters of concrete would be provided over all top reinforcing steel. The same FHPM recommends using higher concrete strengths with lower water and cement ratios to increase density and durability of concrete decks. Both recommendations, combined with the use of epoxy-coated reinforcing steel, should be used when appropriate.

Regardless of the grade of reinforcing steel used or the strength of the deck concrete, it is recommended that stresses in transversely reinforced deck slabs should be limited as follows:

- f_s = 140 MPa **or** f_s = 165 MPa

- f_c = 8 MPa when f'_c equals 20 MPa

  or

  f_c = 0.4 f'_c for a f'_c greater than 20 MPa

Note: Stress limits for the reinforcing steel and concrete (f_s and f_c respectively) are to be determined by Division policy and practice.

Designated, infrequent overloads need not meet the above service limits, however, the slab's ultimate capacity must be more than the factored dead load plus overload with live load factor, B_{LL} = 1.0.

The horizontal railing load shall be treated as an infrequent loading. For ease of design, use the following service limits for dead load plus horizontal rail load for curbs, parapets, overhangs, etc.

- f_s = 165 MPa

- f_c = 0.4 f'_c

Many overload vehicles use tandem axles. Current AASHTO specifications do not provide for wheel distribution and moments with tandem axles. The 1957 AASHTO specifications did include tandem axle effects, and these are shown in Table 10-1. The formulas in this table should be used to design slabs for tandem axle vehicles.

Table 10-1
Slab Live Load Distribution and Moment

Distribution of Wheel Loads	**Main Reinforcement Perpendicular to Traffic Formulas for Moments per Meter Width of Slab**	
	Freely supported spans	**Continuous spans**
Single Axle: Spans 0.61 - 2.13 m, E = 0.6S + 0.76 Spans of more than 2.13 m, E = 0.4S + 1.14	$M = +0.25\frac{P_1}{E}S$	$M = \pm 0.2\frac{P_1}{E}S$
Tandem axles: Spans 0.61 - 2.13 m, E = 0.36S + 0.79 Spans over 2.13 m, E = 0.63S + 1.42	$M = +0.25\frac{p_2}{E}S$	$M = +0.2\frac{P_2}{E}S$

S = effective span length defined under "Span Lengths"
E = width of slab over which a wheel load is distributed
P_1 = load on one wheel of single axle
P_2 = load on one wheel of tandem axle

Slab designs occasionally alternate straight bars in the top and bottom of the slab with bent bars (crankshaft bars). This arrangement provides the design area of reinforcement for the critical tension zones for negative and positive reinforcement and one-half of that amount of reinforcement in the compression zone.

The location of bend points as shown in Figure 10-2 will assure safe design. Bent bars should not be used for decks with flared girders or for curved decks supported by straight girders.
Cantilever slabs should be checked for two loading conditions:

- DL + LL + I
  (Wheel 300 mm from the curb or face of rail.)

- DL + RLL

  Where:

DL	=	Dead load
LL	=	Live load
I	=	Impact
RLL	=	Horizontal rail live load

Stresses for these two loading conditions should be limited as previously noted.

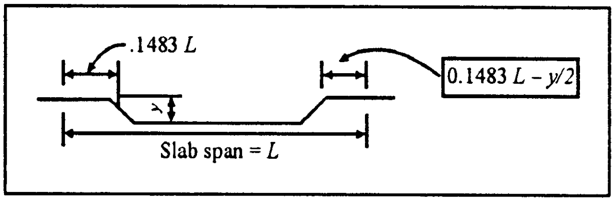

Figure 10-2
Determining Bend Points for
Transverse Deck Slab Reinforcement

2. Rail Design. Bridge railings are an extremely important part of any structure and should be carefully designed and detailed . Railing loads are specified in AASHTO *Standard Specifications for Highway Bridges*, Section 2.7.1.3. and the application of these loads to the deck are covered in Section 3.24.5.2.

The method of connection of rails to decks should allow for ease of deck construction, for alignment, and for ease of rail repair or replacement.

3. Deck Joint Design. The designer should carefully consider accommodating all bridge movements for deck joint designs.

These movements include but are not limited to the following:

- Temperature expansion and contraction.
- Concrete shrinkage and creep.
- Live load rotation.
- Effects of prestressing.
- Foundation movements.

Deck joints should be avoided whenever possible since they are often sources of maintenance problems due to leakage of roadway water and contaminants as well as improper performance.

The following are some rough guidelines for providing for superstructure movements at abutments:

- For flexible abutments (such as a single row of piles with cap) with a pin type connection between superstructure and pile cap, allow 23 meters of contributory movement length. Pile flexibility permits structure movement and deck joints are not required.

- For rigid abutments, and for flexible abutments with more that 23 meters of contributory length, allow superstructure movement to occur against the approach fill, but permit movement between superstructure and abutment with an expansion bearing. Limit contributory length to 46 meters; deck joints are not required.

- For abutments with more than 46 meters of contributory length, provide an independent backwall type abutment with deck joint designed for all movements.

The guidelines above are not meant to be used without careful consideration of bearing protection from contaminants as well as provision for approach fill drainage and abutment details.

Deck joints between abutments are not desirable for the reasons mentioned. In general, they should only be used to separate different superstructure types, relieve frame-type restraint forces, or when the designer feels the provision for movement is critical.

For movements of less than 100 millimeters, the designer can select any of a number of proprietary joints according to the manufacturer's recommendations. It is recommended that on skewed joints, an interlocking type strip or gland seal be used. The joint should be detailed so drainage is properly handled at curbs, sidewalks, parapets, etc. On the plans, the joint width setting at the temperature anticipated during construction should be shown as well as adjustments for other construction temperatures.

For movements more than 100 millimeters, a special design is required.

4. Deck Drains. Every bridge should be analyzed for deck drainage considering width of bridge, superelevation or crown, profile grade, wingwalls, rail type, and geographic location. Consideration should be given to locating bridge deck drains between toes of embankments and installing drainage structures, catch basins, etc., off the bridge.

Deck drains over abutment fill slopes should be avoided. These drains have caused severe erosion on many previous projects. Where deck drains must be provided over abutment fill slopes, the plans must include an erosion control measure to be built at the time of bridge construction.

D. Analysis of Bridge Structures. The analysis of bridge structures begins with an approved TS&L drawing and the AASHTO bridge design specifications. Using these two documents, the design engineer begins by making a preliminary estimate of the members and end conditions. This assumed structure is then analyzed for the design loads and only the critical sections are designed. This design is then compared with the assumed (estimated) sections.

If necessary, the structure is modified and the new structure is again analyzed. This process continues until the optimum design is attained. At this point, the entire structure is designed for all sections and the plans can be produced.

Typically, the design is monitored at each stop for consistency, economic feasibility, and practicality of construction. The designer must never forget the original purpose of the structure and the objectives of the client.

1. Preliminary Sizing and Structure Modeling. The preliminary sizing of the bridge members is aided by previously similar designs as well as the depth-to-span criteria listed under Sections 10.4.E through 10.4.H. This is a critical point in the design process since a wise choice here will reduce the analysis/design iterations mentioned. Experience is invaluable at this stage, so assistance from the FLHD bridge engineer and the senior structural engineers is highly recommended. On certain structures, final design of the deck and traffic rails is now possible. This will help to finalize a portion of the dead load.

Structural analysis is the determination of displacements and stresses due to the known loads. For analysis purposes, the bridge structure must be idealized or modeled as to how the various parts interact to carry the loads to the supports.

In all structural analysis, the following three fundamental relationships must be satisfied:

- Equilibrium.
- Compatibility of displacements.
- Consistency of displacements with the respective stress/strain relationships.

The simplest structure type to analyze is the determinate structure which needs only the equations of equilibrium for complete solution.

The indeterminate structure requires compatibility and stress/strain relationships in addition to the equilibrium equations for complete solution. This requires significantly more effort than the determinate structure.

For each member in the bridge structure, the designer must decide which modeling is appropriate: whether a simplified determinate model will be adequate or whether a more complicated, time consuming indeterminate model is required. For example, a pile cap is often analyzed for 0.8 times the simple span moment to approximate the moments from a more difficult indeterminate solution, and the simple span shears are increased by 20 percent to account for continuity. By contrast, a bridge to be built at a high seismic location must be modeled with a sophisticated three-dimensional mathematical model to permit the required dynamic analysis.

Structure modeling for bridge members and complete structures can only be briefly introduced in this chapter. The inexperienced structural engineer is referred to the many excellent references listed in Section 10.2 as well as professional assistance from the sources listed in the same section.

The engineer should always make certain that the modeling assumptions adequately represent the member's or structures' true behavior for the particular design being conducted.

2. Simplified Methods of Analysis (Hand Method). Before the development of computer structural analysis aids, many techniques for hand analysis were developed. These hand analysis techniques continue to be valuable tools for the structural engineer. These techniques serve to train inexperienced engineers in the structural theory behind the computer programs. They also provide a means to check and understand the results of computer analyses.

Moment distribution is a simple, fast, and accurate method of analyzing continuous girders and frames. It was first taught by Professor Hardy Cross in 1924 and continues to be the bridge engineer's most powerful hand analysis tool. Two excellent references are Section 2 of the *Manual of Bridge Design Practice*, 3rd Edition, by Caltrans and *Moment Distribution*. by J.M. Gere. Moment distribution can easily accommodate the frequent variable moment-of-inertia member types encountered by use of aids for stiffness and carryover factors as well as fixed-end moments for various loadings. The analogous column procedure can be used to develop these for members and loadings not covered by the aids.

For computation of deflections, the moment area and conjugate beam procedures prove very useful. Another deflection computation method which can be extended for calculation of buckling loads and beam-column problems is Newmark's method. These methods are described in *Structural Mechanics* by Samuel Carpenter and *Structural Analysis* by Harold Laursen.

Moments, Shears, and Reactions for Continuous Highway Bridges, by AISC provides complete moments, shears, and reactions for certain continuous beam type members. It provides coefficients for determining influence lines that can be used for both dead and live loads.

The elastic center method can be used to analyze arches and rigid frames. It is described in *Structural Mechanics* by Samuel Carpenter, Section 14 of *Manual of Bridge Design Practice*, 3rd Edition, by Caltrans, and *Analysis of Arches, Rigid Frames and Sewer Sections*, by the Portland Cement Association.

For indeterminate frame type structures, the following procedure for hand analysis has proven helpful:

- From assumed member sizes, calculate stiffness and carryover factors.
- Perform moment distributions for unit fixed-end moments at all member ends individually, and tabulate the results.
- Calculate dead-load and live-load moments and shears at critical superstructure sections using the above unit distributions multiplied by the fixed-end moments for dead and live loads.
- Check the critical superstructure sections for adequacy for the assumed member sizes. (If not adequate, a change at this point will not require much effort.)
- When critical superstructure sections are adequate, design the substructure. (Changes at this point to the substructure members will not waste much previous effort and reanalysis can be done.)
- When substructure design is complete, compute dead load moments and shears for the superstructure at all tenth-point locations.
- Develop and draw influence lines for moments at the tenth points. Live load moments and shears can be obtained semigraphically from these.
- Finally, produce the required envelopes of moments and shears for the completion of the superstructure design.

3. Refined Methods of Analysis (Computer Method). The computer has become an invaluable aid to the bridge engineer. It permits better analysis in much less time than hand methods. It provides the engineer more flexibility to change member sizes and investigate different support conditions, various loading conditions, various modeling assumptions, than possible with time consuming hand analyses.

The computer also allows the engineer to do sophisticated analyses that are much too tedious and time consuming to do by hand. Use of this greater analysis power removes the tedium of hand analysis and allows much more flexibility, but demands that the responsible engineer become familiar with each program, its capabilities and limitations, and verify the results of each analysis. This responsible use of computer tools is essential to maintain professional control of a bridge analysis and design project. The computer can not substitute for an engineer's education, experience, judgment, and responsibility.

It is FLHO policy to encourage the responsible use of state-of-the-art computer tools for analysis and design of bridge structures.

Some recommendations for responsible use of these tool are as follows:

- Determine program authors, original purpose, and history of usage and revisions in order to evaluate the authenticity of reliability, available technical support for and the maturity of the program.

- Obtain complete user documentation as well as sample problem input and output.

- Strive to become familiar with and understand the program's flow and internal algorithms to the greatest extent possible.

- Obtain training and technical support from program authors or experienced users.

- Obtain education in unfamiliar program analysis techniques.

- When using very complicated programs for the first time, obtain a check run from the same program by the author or an experienced user.

- Always correlate the program output results (at least at critical sections) to a rough hand analysis in which you have confidence.

- When reasonable correlation does not exist, determine the cause and pursue better correlation or understanding before using the program further.

- Document helpful notes on input, usage, problem areas, correlation results, etc., for aiding novices and repeat users.

- Avoid becoming overconfident with any program and always verify its results.

A very real danger exists in irresponsible computer usage. Engineers should spend their early career development time learning not just the usage of computer programs, but also the structural theory fundamentals.

In the FLH Divisions, engineers are taught the classical hand analysis techniques described previously along with proper computer usage. Development of these hand skills has shown to provide an excellent theoretical as well as practical application base for the development of responsible bridge engineers.

The following are some excellent computer programs:

- **BDS** (Bridge Design System) is an orthogonal plane frame analysis system applicable to a wide variety of bridges. It has interactive capabilities with Direct Federal CADD equipment.

- **ORG** (Oregon Bridge Analysis Program by the Oregon DOT Bridge Design Section) is a general two-dimensional, finite, element-based analysis program for continuous beam or frame-type bridges. It accommodates standard AASHTO live loadings as well as special user specified vehicles and longitudinal loading, axial strain, and prestress. It reports design oriented outputs of conditions envelopes of shears, moments, and axial forces, influence lines, and stress summaries.

- **SAP** (Structural Analysis Program) is a large, general-purpose, elastic finite element program for static, dynamic and nonlinear analysis.

- **SIMON** is an analysis and design program written by United States Steel for steel plate and box girder bridges. It accommodates standard AASHTO live loads as well as optional-user designated, variable wheel loadings up to 20 axles. Both the working stress and load factor methods are allowed.

- **CURVBRG** analyzes horizontally curved steel girder bridges with a finite element grid- type model. It was developed by the University of California at Berkeley and correlated with actual in-service bridge instrumentation results by Caltrans. It incorporates a very helpful automatic live load generation routine.

- **YIELD** is a reinforced concrete column analysis and design program for biaxial bending and axial load by the ultimate strength theory by Caltrans. It easily accommodates many different column shapes and rebar patterns as well as user input by coordinates to handle virtually any solid or voided column shape and rebar pattern.

- **SEISAB** (Seismic Analysis of Bridges) was specifically written to provide the required dynamic seismic analyses of bridges. It incorporates an input generator to greatly simplify the creation of a three-dimensional model of the bridge. It can accommodate sophisticated foundation modeling when required or the use of many default type conditions. It performs static analyses as well as the AASHTO required single-mode and multimode spectral method dynamic analyses utilizing elastic techniques.

- **STDS** (Segmental Time Dependent System) analyzes the time dependent stresses in segmentally constructed prestressed bridges.

- **TANGO** analyzes steel, concrete segmental, composite, and cable-stayed bridges.

- **M STRUDL** is a linear elastic finite element program for static and dynamic analysis.

- **BRASS** (Bridge Rating & Analysis of Structural Systems) analyzes and designs reinforced concrete box culverts; steel, timber, reinforced concrete, or prestressed girders; and, reinforced concrete piers. The program is a comprehensive system for rating simple and continuous truss and girder floor beam stringer type bridges.

E. Reinforced Concrete Design. Almost every bridge designed in the United States today uses reinforced concrete in some element. This may be footings, substructure elements (such as piers and abutments), retaining walls, girders, decks, or rails. Many bridges consist entirely of reinforced concrete. Since its introduction over 150 years ago, concrete has been the most widely used construction material in the history of civilization. The major advantage in the use of concrete for bridges is its ability to be used in a wide variety of configurations and to have variable content.

1. Structural Types. The following is a list of the more common types of reinforced concrete bridge structures. Each design has distinctive characteristics and attributes.

Reinforced Concrete Slab Bridge.

- *Structural*: The depth-span ratio for simple spans is 0.065± and 0.052 to 0.042 for continuous spans. Solid slabs are used for spans from 5 to 14 meters, cored or voided slabs are used for spans from 12 to 20 meters, and recessed soffit slabs for spans from 12 to 25 meters.

- *Appearance*: Neat and simple; desirable for low, short spans.

- *Construction*: Details and formwork simplest.

- *Traffic*: May be impeded by falsework if cast-in-place due to reduced clearances. Guard-rail should protect falsework openings for traffic lanes.

- *Construction time*: Shortest of any cast-in-place construction.

- *Maintenance*: Very little except at hinges. Future widening may be difficult.

Reinforced Concrete T-Beam Bridge.

- *Structural*: The depth-span ratio for simple spans is 0.07±, 0.065 for continuous spans, 0.080 maximum at supports, and 0.055 minimum at midspan for haunched spans. Smaller ratios are possible, but riding qualities are affected by creep characteristics of concrete. Span range is 9 to 25 meters.

- Appearance: Cluttered in view from bottom; elevation is neat and simple.

- *Construction*: Requires good finish on all surfaces; formwork is complicated.

- *Traffic*: May be impeded by falsework if cast-in-place due to reduced clearances. Guard rail should protect falsework openings for traffic lanes.

- *Construction time*: More than for slabs due to forming, but not excessively long.

- *Maintenance*: Low, except that bearing and hinge details may require attention.

Reinforced Concrete Box Girder Bridge.

- *Structural*: The depth-span ratio for simple spans is 0.06±, 0.055± for continuous spans, 0.02-0.03 at midspan, and 0.05-0.08 at supports for haunched spans. Smaller ratios are possible, but riding qualities are affected by creep characteristics of concrete. High torsional resistance makes it suitable on curved alignment. Span range is 25 to 60 meters. For shorter spans, consider recessed soffit, T-beam, or voided slab bridges.

- *Appearance*: Neat and clean lines from all views; utilities, pipes, and conduits can be concealed.

- *Construction*: Rough form finish is satisfactory on inside surfaces; formwork is complicated.

- *Traffic*: May be impeded by falsework due to reduced clearances. Guard rail should protect falsework openings for traffic lanes.

- *Construction Time*: More than for slabs or T-beams due to staging of concrete placement, but still not excessively long.

- *Maintenance*: Low, except that bearing and hinge details may give some trouble. Future widening may be difficult.

Rigid-Frame Bridges.

- *Structural*: Integral rigid negative-moment knees greatly reduce the positive span moment and overturning moment at foundation level.

  Single rigid portal frames will adapt to narrow water channels, railways, subways, and divided or undivided highways underneath.

  Double-span rigid frames suitable for divided multilane highways underneath with sufficient mall or median width for triple-span support rigid frames (with or without side spans) are possible to accommodate multilane, divided highways with a wider center mall or median.

  Advantage of variable moment of inertia can be easily incorporated. Preliminary proportioning can start with a thickness at the knee equal to approximately twice that at the crown.

- *Appearance*: Graceful and clean; well adjusted to stone facing.

- *Construction*: Usually requiring curved formwork for variable depth.

- *Traffic*: May be impeded by falsework due to reduced clearances. Guard rail should protect falsework openings for traffic lanes.

- *Construction Time*: Similar to that of other types.

- *Maintenance*: Low, except for potential backfill settlement. Limited flexibility for future widening.

Arch Bridges.

- *Structural*: Horizontal reactions created by an arch greatly reduce the otherwise large, positive moment in the center. Constant depth for small spans and variable moment of inertia for medium and long spans. Spans as long as 300 meters have been built. Rise-to-span ratio varies with topography. Thickness at spring lines usually is slightly more than twice that at the crown. Filled spandrels are used only with short spans.

  For medium and long deck spans, open spandrels with roadways carried by columns are the rule. In a through-arch, the center deck usually is carried by hangers and side decks by columns. Use long single spans over deep waterways and shorter multiple spans over wide, shallow waters with rock bottoms.

- *Appearances*: Graceful and attractive, especially over deep gorges, ravines, or a large waterway.

- *Construction*: Either falsework or cantilever methods can be used.

- *Traffic*: When traffic cannot be diverted, the cantilever method may be used instead of falsework.

- *Construction Time*: Usually longer than for other structures. Use prefabricated blocks and post-tensioning when shorter time is desired.

- *Maintenance*: Low.

2. General Requirements and Materials. Concrete to be used for nonprestressed structures will normally have a 28-day compressive strength (f'_c) of 20 to 35 megapascals. The strength required will be based on the member use and product availability from local sources. Poor quality local aggregates often limit the strength of available concrete.

All reinforcing steel should be AASHTO 31M, Grade 60 (400MPa minimum yield strength).

Except in very unusual cases, or in designed splices, reinforcing steel should never be welded.

3. Analysis. All members of statically determinate or indeterminate structures should be designed for the maximum effects of all loads as determined by elastic analysis. Instead of elastic analysis, any acceptable method may be used that takes into account the nonlinear behavior of reinforced concrete, when subjected to bending moments approaching the ultimate. The use of these more exact methods of analysis should be approved on a case-by-case basis by the FLHD bridge engineer.

a. Expansion and Contraction. When designing and detailing reinforced concrete structures, the design engineer should always keep in mind the degree of restraint in members of the bridge. Highly restrained members will almost always crack due to shrinkage or temperature changes. Carefully located construction joints can reduce shrinkage stresses. Stresses due to temperature changes can be controlled by adjusting the stiffness of the structure and by the location of joints.

Creep and shrinkage of concrete are time-dependent deformations and must be included in the design of bridge structures. Short-term loading (live loads) on a concrete bridge induces elastic deformations. Dead loads or superimposed dead loads, however, are long-term effects that must be considered.

Creep of concrete is the phenomenon in which the deformation continues with time under constant load. This response can be related to the initial elastic deformation or strain as determined by the following equation:

$$C_t = [\frac{t^{0.60}}{10 + t^{0.60}}]C_u$$

where: $C_t = \dfrac{creepstrain}{nitialelasticstrai}$

$C_u = 2.35$

t = time (days)

Creep is also represented by the curve shown in Figure 10-3.

Shrinkage is defined as the volume change in the concrete with respect to time. The associated shrinkage strain can be computed from the following formula:

$$E_{sh} = \frac{t}{35 + t}(E_{sh})u$$

$$(E_{sh})u = 800\ (10^{-6} mm\ mm^{-1})$$

Shrinkage strain is also represented by the curve shown in Figure 10-4.

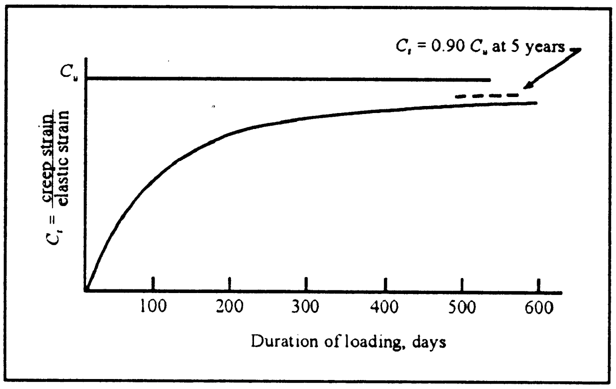

Figure 10-3
Standard Creep Coefficient Curve

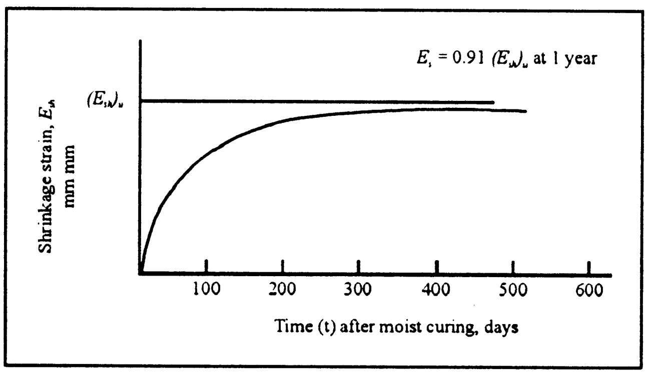

Figure 10-4
Standard Shrinkage Strain Curve

b. Stiffness and Frame. The following guidelines concerning frame analysis with concrete columns, walls, or single shaft piers may be used as a guide to supplement the AASHTO specifications.

For figuring stiffness, include the moment of inertia of the reinforcing steel for all substructure units and use the gross concrete moment of inertia for superstructure units.

The effective height of a pier column or wall should be taken from the centroidal axis of the superstructure to the bottom of the footing or to the top of the footing if an articulated hinge is used. Also, the substructure frame characteristics shall be based on the concrete below the superstructure soffit. If there is a monolithic cap that extends below the superstructure soffit by more than 10 percent of the effective height, the effect of such a cap shall be included in the frame characteristics. The effect of stiffness of footings shall be ignored. In the cases where articulated hinges are used, the moment due to thrust shall be used in figuring footing pressures for such footings.

For stiffness of skewed wall piers up to 20° skew, use the moment of inertia about the axis of the wall. For skews greater than 20°, careful analysis should be made on a case-by-case basis to determine the proper substructure stiffness.

The use of articulated hinges at the bottom of the columns will be restricted to the cases when the size of resulting piers and footings are excessive. When no articulated hinge is used, the pier shall be assumed hinged at the bottom of the footing for allowable soil pressure up to 300 kilonewtons per square meter (300 kilopascals). In this case, proportion the footing area for the uniform footing pressure. Design the footing itself (depth, reinforcement, and shear) for a nonuniform pressure by applying 20 percent of the moment at the column top to the footing.

For allowable soil pressure greater than 300 kilonewtons per square meter (300 kilopascals) and up to 450 kilonewtons per square meter (450 kilopascals), the pier shall be assumed to be 50 percent fixed.

For allowable soil pressure greater than 450 kilonewtons per square meter (450 kilopascals), the pier shall be assumed 100 percent fixed.

The use of a single row of foundation piles shall be considered as a hinge at the bottom of the footings, while the use of more than one row of foundation piles shall be considered as fully fixed at the bottom of the footings.

For skewed walls or single shaft piers, the longitudinal and transverse moments from superstructure (due to any load) should be resolved into components normal and parallel to the axis of the pier.

For circular shafts, the moment should be combined into one resultant depending on the magnitude of the moments.

In nonskewed bridges, the shear load from a span is distributed uniformly into a support by assuming each girder carries its own portion. In a skewed bridge, the load tends to distribute to the supports in a direction normal to the support. This causes a greater portion of the load to be concentrated at the obtuse corners of the span and less at the acute corners.

A graph has been developed to provide adjustment factors for applied shears calculated without considering skew effects. This graph, and examples for its use, are available in the bridge design offices of the Federal Lands Highway Divisions.

For curved bridges having skews greater than 45°, the design should consider a more exact analysis, such as three-dimensional computer programs, that consider torsion.

4. Design. *AASHTO Standard Specifications for Highway Bridges* now permits the designer to choose one of two methods for proportioning reinforced concrete bridge members:

(1) "Service Load Design Method (Allowable stress design)", AASHTO Section 8.15, or

(2) "Strength Design Method (load factor design)", AASHTO Section 8.16.

Service Load Design is a modification of the method used for many years, sometimes called working stress design, since stresses calculated for service load effects are controlled within specified allowable, or working stresses. Strength Design, sometimes called ultimate strength design or load factor design, is based upon providing a member strength sufficient to carry loads that are specified multiples of the service loads.

Exhibit 10.1, at the end of this chapter, contains selected portions of a paper titled *Design Methods and Strength Requirements*, by Ashby T. Gibbons, Jr. of PCA. It is offered to give the design engineer a brief history of these two design methods and to assist in deciding which method to use for design.

It is only a matter of time before service load design will be eliminated from the AASHTO specifications. (The ACI Code did so in 1977 and included it as an *alternate design method* in the appendix of the code.) The use of strength design is highly encouraged for all FLH projects.

5. Specifications, Design Aids, and Policies. In each design office, there are specific policies and design aids that clarify, modify, and guide the usage of the AASHTO specification. These are voluminous and are updated frequently to keep up with the AASHTO yearly interim specifications. The interims are necessary since bridge design is dynamic in nature (i.e., research and development of new technologies force changes in both design specifications as well as construction methods).

Since these policies and aids are voluminous and are frequently updated, it is impractical to include them in this design manual. The designer is referred to the FLH Division Bridge Engineer and Senior Structural Engineers for these policies and aids.

F. Structural Steel Design. Although true structural steel was used for the eye-bars of suspension bridges in the early 1800's, it was not until about 1870 that the first all steel bridge was constructed. Today, there is a wide variety of steels available for bridge design. The bridge engineer needs to have a working knowledge of the physical properties of these steels in order to make a proper selection.

1. Structural Types. The following is a list of the more common types of structural steel bridges. Each design has distinctive characteristics and attributes.

Composite Wide Flange Beam.

- *Structural*: This structure type has low dead load which may be of value when foundation conditions are poor. Depth-span ratio for simple spans is 0.060. Can be used with timber decks for low volume structures or pedestrian bridges. Suitable for spans up to 25 meters. Larger sizes of wide flange beams may not be available in many areas.

- *Appearance*: Can be attractive. Best for simple spans.

- *Construction*: Details and form work simple. Partial length cover plates welded to bottom flange will improve economics.

- *Traffic*: Minimal traffic problems; limited to short periods of time for erection and installation of protection nets if required.

- *Construction Time*: On the job, very short, but procurement of steel may cause delay.

- *Maintenance*: Painted steel structures require routine maintenance depending on environmental conditions. Weathering steel reduces maintenance. Weathering steel should be carefully considered in desert climates, coastal areas, or in areas subject to heavy use of deicing salts. Weathering steel may cause staining of concrete piers and abutments.

Composite Welded Girder.

- *Structural*: This structure type has low dead load, which may be of value when foundation conditions are poor. Depth-span ratio for simple spans is 0.060 and 0.045 for continuous spans. Can be adapted to curved alignment. Suitable for spans 18 to 90 meters. May be competitive when an erected type of superstructure is required. Competitive with precast concrete girders.

- *Appearance*: Can be made to look attractive. Girders can be curved to follow alignment.

- *Construction*: Details and formwork simple. Transportation of prefabricated girders may be a problem.

- *Traffic*: Same as for composite wide flange beam.

- *Construction time*: Short time on the project, but procurement and fabrication of steel may cause delay.

- *Maintenance*: Same as for Composite Wide Flange Beam.

Structural Steel Box Girder.

- *Structural*: Use multiple boxes for spans up to 60 meters and single box for longer spans. Use depth-span ratio for continuous spans of 0.045 and 0.060 for simple spans. Usable for spans 18 to 150 meters. More expensive than steel "I" girder. More economical in the upper range of usable span and where depth may be limited.

- *Appearance*: Generally pleasing. Better than steel or precast concrete girders.

- *Construction*: Very complicated welding and welding details. Because of the many opportunities for welding and detail errors that can give rise to fatigue failures, the steel box should only be used in very special circumstances.

- *Traffic*: Erection requires substantial falsework bents at splice locations.

- *Construction Time*: Procurement of steel and extensive fabrication requires considerable time.

- *Maintenance*: Same as for composite wide flange beam.

Steel Railroad Structure.

- *Structural*: Reinforced concrete deck preferred. Steel plate deck may be used. Deck type structures are more economical than through girder structures. Depth-span ratio is 0.10 for deck type (not including the 0.61 meters from top of rail to bottom of ballast). Through girder structures requires substantial deck thickness from top of rail to bottom of ballast. Depth-span ratio of through girders is 0.13.

- *Appearance*: Can be attractive.

- *Construction*: Details simple. Shop fabricated.

- *Traffic*: Minimal traffic problems.

- *Construction Time*: Same as for Composite Welded Girder.

- *Maintenance*: Same as for composite wide flange beam.

2. General Requirements and Materials. Steel structures shall be designed using the following:

- *ASTM A 36M (AASHTO M 183M) Structural Carbon Steel*. This is the "basic" steel to be used for painted structures where high strength is not required.

- *ASTM A 572M, Grade 345 (AASHTO M 223M) High Strength, Low-Alloy Structural Steel*. This steel is to be used for painted structures where high strength is advantageous.

- *ASTM A 588M (AASHTO M 222M) High Strength, Low-Alloy Structural Steel*. This steel (sometimes referred to as weathering steel) has an increased resistance to corrosion and shall be used for unpainted structures.

The use of structural steel conforming to ASTM A 514M or A 517M should be restricted to only very unusual structures. These very high strength, quenched, and tempered alloy steels require expert design and extreme controls for fabrication due to their critical fatigue and fracture characteristics. Their use on projects will be very rare.

The use of structural steel for main member tension material in excess of 50-millimeter thickness should be avoided.

In general, bolts for structural steel bridges shall be fabricated from ASTM A 325M (AASHTO M 164M) steel - Either type 1 or type 2. ASTM A 325M, Type 3 bolts with weathering characteristics are compatible with ASTM A 588M structural steels.

The use of high strength ASTM A 490M (AASHTO M 253M) bolts should be severely restricted. Like very high strength structural steels, these bolts require extreme construction control.

3. Design. In the past, steel bridge design was relatively simple. Usually, the structure was only required to span an obstacle by the simplest and most direct means. Material stresses were kept quite low. Today, however, steel bridges are required to match the highway alignment, which often results in curved structures. Economic considerations require the use of steels to their maximum, resulting in very high material stresses. This means that the design details for steel bridges are of utmost importance. Current specifications become very complex and should be carefully adhered to for all steel bridge design.

a. Fatigue and Stress Considerations. The fatigue provisions of the AASHTO Design Specifications were developed from research and studies of failures in the field. Details for main load-carrying members, such as cover plates and butt weld of tension flanges, are familiar to bridge designers. The effects of connection to the main members, however, are not as familiar and have been a source of more and more fatigue problems in recent years.

A recent change in the specification is the stress range concept. Research by Dr. John Fisher at Lehigh University has shown that the range of stress is the single most damaging factor in fatigue. Thus, only the live load fluctuation is significant and the stress level does not significantly influence fatigue behavior.

The type of loading, stress category (connection details), and redundancy control the allowable stress range.

(1) Type of Loading. The number of cycles of fatigue life depends on the type-road and the type of live loading. Many projects are low-volume roads in terms of truck traffic and should be designed for 100 000 cycles of loading. There are some projects, (such as major forest highway routes), that may require a design for 500 000 or even 2 000 000 cycles of loading. The number of loading cycles should be determined on a case-by-case basis.

(2) Stress Category. There are several stress categories, A through F. Each of these categories is described in the tables and illustrative examples are shown in Chapter 10, Section 10.3 of the Division I - Design of the AASHTO Standard Specifications for Highway Bridges. Plain plates are in category A and the most severe connection details are in Category E.

(3) Redundancy. Bridge structures where the failure of a member of a single element could cause collapse of the structure (such as a single box girder, truss, etc.) are considered nonredundant structures, and a more severe restriction has been placed on them by shifting the allowable stress range by one loading cycle. This reduction to a lower stress range makes details that fall in category E very uneconomical and, in essence, restricts their use.

(4) Charpy V-Notch Impact Requirements. Main load-carrying member components subjected to tensile stress require minimum supplemental impact properties. These impact requirements vary depending on the type of steel used and the average minimum service temperature to which the structure may be subjected. For service temperatures below -35°C, special values are required.

The basis and philosophy used to develop these requirements are given in a paper entitled *The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels* by John M. Barsom, February 1975, available from the American Iron and Steel Institute, Washington, DC.

(5) Identification of Main Member Components Subject to Tensile Stress. Main load-carrying member components subject to tension or reversal of stress must be identified on the contract drawings as well as the appropriate temperature zone. These are necessary to specify the required charpy V-notch impact properties and the required nondestructive testing of groove welds.

The designer should consult the Federal Lands Highway Division Bridge engineer or a Senior Structural engineer for guidance.

(6) Fracture Control Plan (FCP). The FCP is an AASHTO guide specification for fracture critical nonredundant steel bridge members. The FCP was devised to prevent collapse of steel bridges. Much of the FCP relates to design, welding, and material properties.

The designer is responsible to designate any member or structural component as a fracture critical member (FCM) when failure of the FCM would cause the structure to collapse. The FCP requires the FCM to be fabricated in a qualified shop and inspected by qualified inspectors; requires nondestructive inspection (NDI) by qualified testers; and supplements the current AWS and AASHTO welding specifications, and specified material toughness.

The FCP is a comprehensive plan whose adoption should improve the overall quality of steel structures from design through fabrication.

An excellent source of guidance on fatigue is the *Bridge Fatigue Guide, Design and Details* by Dr. John W. Fisher and published by the American Institute of Steel Construction.

b. Efficient Girder Depths. The first step in the design of a steel bridge is to determine the most efficient web depth. The determination of this depth is based on several parameters. Girders may be classified as either symmetrical or unsymmetrical as shown in Figure 10-5.

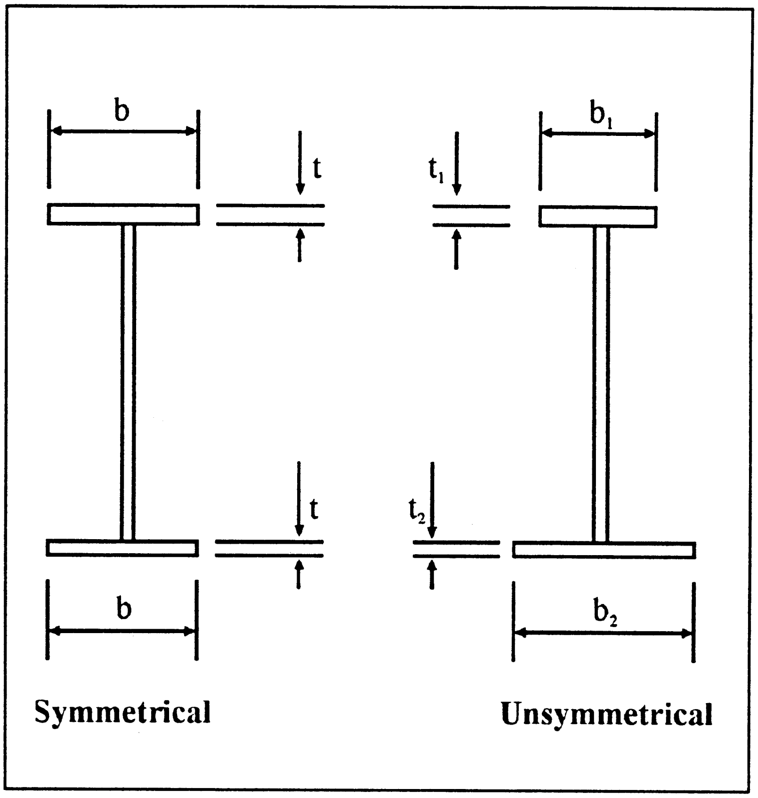

Figure 10-5
Girder Classification

In addition to being classified as symmetrical or unsymmetrical, steel members can be further categorized as follows:

- Compact.
- Noncompact.
- Braced.
- Unbraced.
- Transversely stiffened.
- Longitudinally stiffened.

The steel girders designed by the FLHD Bridge staff are usually welded plate girders. Typically, these are noncompact, unsymmetrical, and transversely stiffened. They can be either braced or unbraced, and are frequently stiffened longitudinally for longer span girders.

The efficient depth is calculated in order to maximize the structural efficiency for the minimum weight. The depth can be approximated from the following formula:

$$d_{ew} = \sqrt[3]{1.5 s \left(\frac{d}{t_w}\right)}$$

where:

d_{ew} = efficient depth of web

s = required section modulus from m/f_b

m = design moment and f_b = allowable bending stress

$\frac{u}{t}$ = depth-to-thickness ration for web

For a complete discussion of girder depths, see Section 4.2 of *Design of Welded Structures*, by Omer W. Blodgett.

c. Deflection. In addition to providing camber to compensate for deflection due to dead load and for vertical curvature required by the profile grade, additional camber should be provided to compensate for deflection due to shrinkage of the deck concrete. This is important for spans greater than approximately 30 meters. The following procedure describes the method to be used to estimate this deflection. (See Figure 10-6 for values of "e" and "t").

Force due to shrinkage:

$$F = 0.0003(A_c)(E_c)$$

$$M = F(e + t/2)$$

Where:

Ac = area of concrete slab
E_c = modulus of elasticity of concret

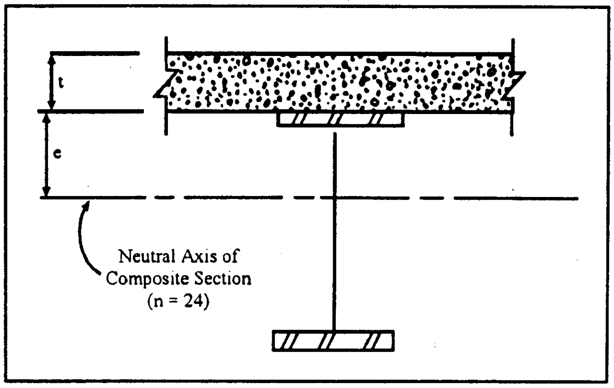

Figure 10-6
Deflection Due to Deck Shrinkage

(1) Shrinkage in Simple Beams. Calculate moment due to shrinkage and apply as a uniform load. Find deflections due to shrinkage using composite section for moment of inertia.

(2) Shrinkage in Continuous Beams. It is assumed that the shrinkage force in the slab takes place when the slab is in compression, such as between points of inflection in interior spans and between end pier and inflection points at end spans.

To calculate deflections, do the following:

1. Find uniform moments along the beam in compression zones (between points of inflection)

2. Calculate fixed end moments together with distribution and carryover factors.

3. Combine 1 and 2 for final moments and find deflections due to these moments using respective moments of inertia.

d. Splices. Field splices should be bolted splices. Field welded splices of primary structural members should be avoided.

e. Diaphragms, Cross Frames, and Lateral Bracing. The AASHTO specifications now include provisions to determine the need for lateral wind bracing for the bottom flanges of rolled beam and plate girder bridges.

The application of these provisions to include the effects of wind loads plus dead and live loads is covered in the pamphlet *Determining the Need for Lateral Wind Bracing in Plate Girder Bridges*, a supplement to Chapter 5, Volume II, USS Highway Structures Design Handbook.

Lateral wind bracing should not be included if not required by stresses.

f. Composite Deck Design. Concrete deck slabs should be made composite with steel girders for the entire length of simple spans and for the positive moment regions only on continuous spans. This is generally achieved with welded stud shear connectors.

4. Specifications, Design Aids, and Policies. Each design office has specific policies and design aids that clarify, modify, and guide the usage of the AASHTO specifications. These are voluminous and are updated frequently to keep up with the AASHTO yearly interim specifications. The interims are necessary since bridge design is dynamic in nature; i.e., research and development of new technologies force changes in both design specifications and construction methods. Since these publications are voluminous and are frequently updated, it is impractical to include them in this design manual.

The designer is referred to the FLHD Bridge Engineer and Senior Structural Engineer for these policies and aids.

G. Prestressed Concrete. The first prestressed concrete bridge in the United States was constructed in 1949. Since 1960, most bridges in the United States with a span range of 18 to 36 meters have been constructed with prestressed concrete. Since the late 1970's, post-tensioned continuous or cantilever bridges with spans of 45 to 200 meters have gained in popularity.

1. Structural Types. The following list of features of the more common structures provides information to assist in the preliminary selection and sizing of members.

Cast-in-place concrete slab.

- *Structural*: Used for spans up to 20 meters. Recommended for conditions where very low depth-span ratio is required. Can be used for either simple or continuous spans. The depth span ratio is: 0.030 for simple and continuous spans. More expensive than reinforced concrete slabs.

- *Appearance*: Same as reinforced concrete slabs.

- *Construction*: More difficult than reinforced concrete slabs.

- *Traffic*: May be impeded by falsework due to reduced clearances. Guide rail should protect falsework openings for traffic lanes.

- *Construction Time*: Shortest of cast-in-place construction; longer than precast slabs.

- *Maintenance*: Very little.

Precast concrete slab.

- *Structural*: Standard plans for corded slabs of spans 6 to 15 meters are available. Not recommended for long multispan structures because of difficulties in camber control resulting in undesirable riding qualities. Economical where many spans are involved or in areas remotely located from concrete batch plants.

- *Appearance*: Same as reinforced concrete slab.

- *Construction*: Details and formwork very simple. Shop fabrication methods employed.

- *Traffic*: Very little interference except during erection.

- *Construction Time*: On site, very short. Very little time required for plant fabrication.

- *Maintenance*: Very little.

Precast Concrete Girder.

- *Structural*: Applicable to spans 9 to 43 meters. Standard sections are available for "I" girders and many other girder shapes to cover complete range of spans. Design analysis to determine prestress force, concrete strength, and camber. Structure depth is girder depth plus necessary slab thickness. Girders longer than 36 meters cannot be hauled over many highways. Depth-span ratio is: 0.055 simple spans, 0.050 continuous spans. Competitive with steel girders, and very economical in areas near precasting plants.

- *Appearance*: Similar in appearance to T-beams. Straight girders on curved alignment look awkward.

- *Construction*: Require careful handling and transporting after fabrication. Fabrication plants nationwide cast a wide variety of sections in addition to standard AASHTO sections.

- *Traffic*: Same as prestressed slabs.

- *Construction Time*: Same as steel girders. Fabrication may require additional time.

- *Maintenance*: Very little except at hinges or joints.

Cast-in-place box girder (post-tensioned).

- *Structural*: Requires detailed stress analysis. Depth-span ratio: 0.040 continuous spans, 0.045 simple spans. High torsional resistance makes it desirable on curved alignment. Dead load deflections minimized. Desirable for simple spans over 30 meters. Long-term shortening of structure must be provided for. About the same as conventionally reinforced box girder. Used for spans up to 180 meters.

- *Appearance*: Better than conventional box girder because of shallow depth. Has all other qualities of conventional box girder. Excellent in metropolitan areas. Can be used in combination with conventional box girders in long structures with varying span lengths to maintain constant structure depth.

- *Construction*: Same as conventional box girder.

- *Traffic*: May be impeded by falsework due to reduced clearances. Guard rail should protect falsework openings for traffic lanes.

- *Construction Time*: Longest for any prestressed concrete structure due to delay before tensioning is allowed to proceed.

- *Maintenance*: Very little except at joints or hinges.

2. General Requirements and Materials. Concrete in prestressed members is subject to higher stress levels than concrete in nonprestressed, reinforced members. Therefore, on all projects under the jurisdiction of FLH, the minimum compressive strength at the time of initial prestress shall be as follows:

- f'_{ci} = 25 megapascals (post-tensioned members).

- f'_{ci} = 30 megapascals (pretensioned members).

Prestressing steel strands are available in diameters from 6.4 millimeters to 15.2 millimeters, in grades of 1.72 gigapascals or 1.86 gigapascals , and as either stress-relieved or low-relaxation. The grade or strand indicates the ultimate strength and the type of strand (i.e., stress-relieved or low-relaxation) and defines the manufacturing process and prestress loss characteristics.

The most common strand used on FLH projects is 12.7 millimeters diameter, grade 1.86, low-relaxation strand. However, some localities may continue to use stress-relieved strand. The specifications should allow a prestressing firm to change the type of strand if desired. Any changes should be redesigned by the manufacturer and checked by the Government. These changes should be shown on the fabrication plans and submitted for approval.

Prestressing steel strands with a 15.2 millimeters diameter should not be used for any precast, prestressed member.

Properties and strengths of seven-wire, grade 1.86 strand are shown on Table 10-2.

Table 10-2
Properties of Prestressing Strand

Seven-Wire-Strand					
Nominal Diameter, mm	9.5	11.1	12.7	14.3	15.2
Area, mm^2 (A*s)	54.8	74.2	98.7	123.9	138.7
Mass, kg/m	0.43	0.60	0.79	0.97	1.12
0.7$f_s'A^*_s$, kilonewtons	71.6	96.5	128.6	161.5	181.0
0.75$f_s'A^*_s$, kilonewtons	76.5	103.6	137.9	173.0	193.5
0.8$f_s'A^*_s$, kilonewtons	81.8	110.3	146.8	184.2	206.8
$f_s'A^*_s$, kilonewtons	102.0	137.9	183.7	230.4	258.4

Note: f_s' = Ultimate strength of 1.86 GPa

3. Analysis. Stresses are introduced into the concrete opposite to the stresses resulting from loads acting on the structure. The stresses are introduced in such a manner that allowable working stresses will not be exceeded. Compressive stresses are induced in the face of the member where tensile stresses tend to develop due to loads. These induced stresses result from a compressive force that is transmitted to the concrete from the prestressing steel.

Prestressed concrete makes full use of the compressive strength of the concrete and the tensile strength of the prestressing steel. Ordinary reinforced concrete does not use the concrete to its full advantage. For comparison purposes (using the same allowable concrete stress), see Figure 10-7. The resulting moment for the reinforced concrete section is calculated in the normal manner, and the resisting moment shown for the prestressed section is approximately the net resisting moment for applied loads after prestressing.

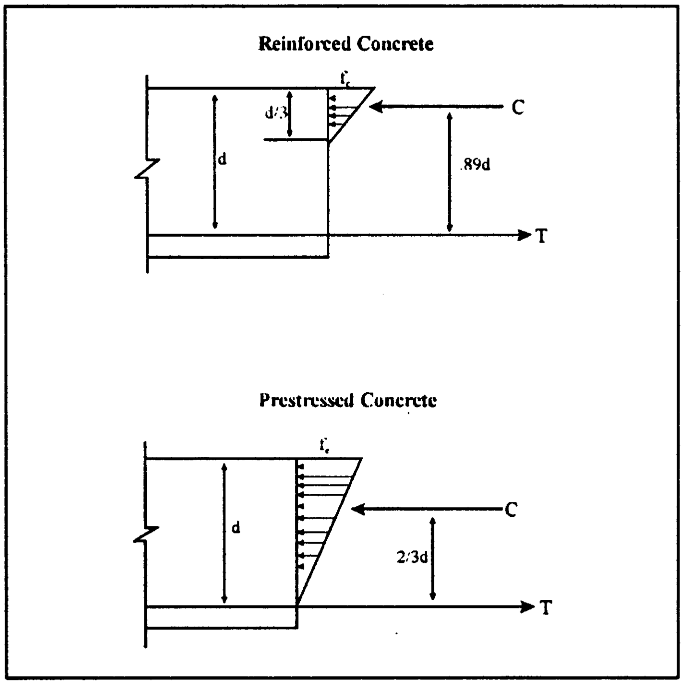

Figure 10-7
Concrete Beam Stresses

As shown in Figure 10-7, a prestressed section with beams of the same depth can resist over twice the moment that the reinforced concrete section can resist. Furthermore, the allowable working stress can be doubled for the prestressed section, thus making the resisting moment over four times that of the reinforced concrete section. The prestressed section makes use of the entire concrete area; however, the reinforced section uses about 1/3 of the area, 2/3 being used to hold the reinforcing steel away from the working section, resist shearing stresses, and develop the bond between the concrete and reinforcing steel.

Two advantages of a prestressed concrete structure are the reduction of both concrete and steel quantities. Other advantages of a prestressed concrete structure are as follows:

- A considerable reduction in depth of section, not only relative to reinforced concrete, but also relative to structural steel.
- A reduction in the cracking of concrete within a known range of load. This results in greater durability under severe conditions of exposure.
- A prestressed structure that has maximum rigidity under working loads and maximum flexibility under excessive overloads.
- The capacity to support a load in excess of the design load in which cracks appear but disappear completely on removal of the excess load.
- A definite reduction in diagonal tension. An important factor in reinforced concrete but often less severe in prestressed concrete.
- Use of pretested structural materials. During the prestressing operations, the steel is tested to a stress that will never again be reached under design loads. The same applies to the concrete, in many cases. This inplace testing is impossible in ordinary reinforced concrete structures.
- Added flexibility for construction.

There are two methods of applying a prestressing force. *Pretensioning* is tensioning of the steel that is done before the concrete is cast in the forms. *Post-tensioning* is tensioning that is done after the concrete has been cast and has attained the required strength. In the former, the force is transmitted by the bond between the steel and concrete. The initial prestress is immediately reduced due to the deformation and shrinkage of the concrete. Gradually, these losses are increased by further shrinkage and creep of the concrete. In post-tensioning, the elastic shortening losses are lower than in pretensioning. Like pretensioning, there is a gradual loss due to the shrinkage and creep of the concrete and the creep of the steel. Consequently for equivalent members, the pretensioning method requires a greater initial prestressing force to compensate for the larger losses.

Pretensioning is practical only within factory or mass production facilities, since permanent anchorages are required to take the reaction of the stressed wires until the concrete attains the required strength.

Several methods of stressing and anchoring "post-tensioned" steel are in use. The methods used most commonly in the United States at present are illustrated in the *Post-Tensioning Manual*.

Design prestressed concrete members to meet the following requirements:

- The prestressing force shall be determined by allowable stress design for service level loads using unfactored MS loads.

- The ultimate moment capacity should be checked by *load factor design* using the ultimate strength theory for factored loads (MS load or design overload, whichever is greater).

- Shear design should be based on strength (load factor design) using the ultimate strength theory with factored MS load or design overload, whichever is greater.

The following assumptions shall be made:

- Strains vary linearly over the depth of the member throughout the entire load.

- Before cracking, stress is linearly proportional to strain.

- After cracking, tension in the concrete is disregarded.

Load factors are multiples of the design load applied to the structure to ensure its safety and are represented by the following formulas:

MS Loads:

$$M_{UA} = \frac{1.3}{\phi}\,[D = \frac{5}{3}(L = I)_{MS}] + 1.0M_S$$

Design Overloads (OL):

$$M_{UA} = \frac{1.3}{\phi}\,[D + (L+I)_{OL}] + 1.0M_S$$

where

M_{UA}	=	Ultimate moment applied
M_s	=	Secondary moment due to prestress force
D	=	Dead load
L	=	Live load
I	=	Impact
ρ	=	Section strength factor 1.00 for precast members 0.95 for cast-in-place, post-tensioned 0.90 for shear, torsion

Expansion and contraction are important design parameters and require consideration. Bearings and joints for prestressed bridges must accommodate the movement from prestress shortening in addition to temperature changes. In framed structures, the stresses resulting from these movements must be included in the design.

The prestress shortening to be expected can be calculated by use of the equation:

$$\Delta = \frac{PL}{AE}$$

where:

P = total prestressing force
L = one half of the length between piers
A = cross sectional area of the superstructure
E = elastic modulus of superstructure concrete

4. Specifications, Design Aids, and Policies. In each design office, there are specific policies and design aids that clarify, modify, and guide the usage of the AASHTO specifications. These are voluminous and are updated frequently to keep up with the AASHTO yearly interim specifications. The interims are necessary since bridge design is dynamic in nature; i.e., research and development of new technologies force changes in both design specification as well as construction methods. Since these policies and aids are voluminous and are frequently updated, it is impractical to include them in this design manual. The designer is referred to the FLHD bridge engineer and senior structural engineers for the policies and aids.

H. Timber. Timber bridges, properly designed and treated with modern preservatives, will give many years of minimal-maintenance service. Their use is normally limited to low-volume, secondary-road bridges and pedestrian bridges.

The following are the most common bridge components that use timber:

- Piling
- Beams or girders
- Decks
- Rails and posts

As can be seen, it is possible to construct entire bridges of timber, however, this is rarely done. Rather, timber is combined with other elements, such as steel girders, concrete substructures, etc., to produce the most economical and maintenance-free structure possible.

With the exception of temporary structures, all exposed timber should be pressure treated. The most common species of timber used are Douglas Fir and Southern Pine. Hardware is normally galvanized.

1. Substructures. Timber pilings are displacement piles that normally function as friction piles. When used as point bearing piles, or when difficult driving conditions are encountered, reinforced pile tips should be considered. AASHTO Section 4.5.7 gives guidance on the design of timber piles as a structural member. Timber piling should not be used in soils where large boulders or cobbles exist. Timber piling is most economical when used for relatively shallow foundations. Timber-pile bents should not be used in streams that carry heavy draft and debris.

2. Superstructures. The most common type of timber superstructure is the longitudinal girder, simple-span bridge. Straight girders are most economical for short spans of 6 to 18 meters. Spans up to 30 meters are possible using glue laminated girders, but may not be economical depending on location, live load, and vertical clearance requirements.

A second common type of timber superstructure is the truss bridge. This may be either a bowstring truss or a parallel-chord truss. Bowstring trusses are of two general types: the pony truss for spans from 15 to 30 meters, and the through span truss for spans longer than 30 meters. Commonly, top and bottom chords are glue-laminated members and web members are sawn timber. Steel rods are used in tension members in the web. When water clearance allows, the parallel chord truss may be used as a deck span, thus enabling pier heights to be greatly reduced. The parallel chord truss may also be used in a through span. The practical span range for either system is 30 to 75 meters.

For long, clear-span timber bridge construction, the deck-arch bridge has been used. With this type of construction, pier height is held to a minimum and yet the bridge is well above high water. The deck arch is practical for spans from 18 to 90 meters, and is particularly suitable when rock canyon walls can reduce the foundation sizes for arch abutments. All present designs for timber-arch bridges should specify laminated construction.

3. Decks. Timber decking is the most common use of timber in current bridge construction. In the past, the most common type of deck was nail-laminated decking using nominal 50 millimeter dimension lumber fastened with through-nailing of the laminations and toe-nailing of the laminations to longitudinal stringers.

Today, most decks are glue-laminated timber systems that allow longer deck spans. These glue-laminated deck systems are plant fabricated in panels and may be designed for either transverse or longitudinal decking. This has necessitated the development of improved connection systems to connect the deck panels to the superstructure. Many current systems are detailed in timber design publications and manuals. The designer should carefully select and analyze these connection systems for strength, ability to resist shrink or swell of timber members, and for resistance to loosening due to vibration or deflection.

Few deck connection systems provide true lateral support to the compression flanges of supporting girders. This lateral support must be considered partial lateral support at diaphragms or cross-frames in most cases.

Most timber deck designs should include wearing planks (sometimes called running planks) to protect the primary decking from tire wear. These wearing planks are nailed to the lower deck and are replaced as required.

All timber decking should be pressure treated to extend decking life.

Timber decks may also be protected by an asphaltic concrete overlay. Detailed mix designs for these overlays are available from the American Institute of Timber Construction.

For some designs, this overlay can be used to provide a crown section for roadway drainage. It is not practical to crown decks constructed entirely of timber.

4. Rails and Posts. Timber rails and posts are commonly used for railings on pedestrian bridges due to the ability of timber to produce an aesthetically pleasing appearance. These railings normally use glue-laminated timber for the rails and either glue-laminated or sawn timber for the posts. Do not use creosote treated timber for pedestrian rail systems.

Timber rail systems, intended for vehicular traffic, almost always incorporate a heavy timber or concrete curbing and a steel backing plate for the timber rail elements. The rails are usually glue-laminated timber and posts may be either glue-laminated or sawn timber. As with all rail systems, timber railings must be carefully designed with particular attention paid to connection, joints, and splices.

I. Bearings. Bridge bearings serve several purposes, the first of which is to transmit vertical loads from the superstructure to the substructure. These bearings must also transmit lateral forces, longitudinal or transverse in direction, between the superstructure and the substructure. In addition, these bearings should take care of girder rotation.

Fixed (sometimes called pinned) bearings transmit both longitudinal and transverse forces. Expansion bearings generally transmit only friction or longitudinal shear forces from the movement of the bearing during longitudinal expansion or contraction. Expansion bearings also usually transmit transverse lateral forces from the superstructure to the substructure.

Numerous types of bearings are available. The following are the most common bearings in current use:

- *Elastomeric bearing pads* come in several configurations. Plain pads of 12-millimeter thickness are used for fixed bearings in conjunction with lateral-load transfer devices such as keys in construction joints, shear lugs, or anchor bolts. These 12-millimeter pads allow rotation of girders and provide distribution of loads between slightly uneven bearing surfaces.

- *Laminated pads* of thicknesses up to a maximum of about 100 millimeters are used for expansion bearings. These bearings may have either steel shims or fabric shims, usually spaced at 12-millimeter increments. Laminated pad bearings usually are used with transverse lateral load transfer devices. They are usually designed for horizontal movement in one direction only.

- *Steel-shim laminated bearings* with other than integrally molded edge protection have been found to be unsatisfactory in use. Laminated bearings not molded as a single unit under heat and pressure are susceptible to bond failure between the layers.

The durometer hardness of the elastomer should be specified on the plans. This hardness should be based on the lowest anticipated service temperatures.

Geometric proportions (shape factors) are given in the specifications to ensure stability of the bearings. Bearing design is controlled by compressive stress, shape factor, hardness, and compressive strain. Bearing thickness is controlled by movement requirements.

- *Sliding bearings* are a configuration normally consisting of a combination of a thin elastomeric pad (to allow rotation and to control the distribution of the bearing loads), steel bearing plates, and a TFE (teflon) surface moving against either another TFE surface or against a stainless steel surface. These bearings have very low friction valves. They are used for moderate-span steel structures. Lugs to transfer lateral forces are often incorporated into the design of bearing.

- *Pot bearings* are bearings used for longer span bridges in which the reactions could be 1.33 meganewtons or greater. They incorporate an elastomeric material-confined material. The fluid-type action distributes the load evenly on the base plate. These bearings may be designed with TFE sliding surfaces to allow movements.

- *Roller and rocker bearings* are bearings generally used for longer-span steel bridges. They are normally either painted or galvanized, even when used with weathering steel superstructures. Small diameter rollers do not perform satisfactorily due to corrosion and should not be used. These bearings can be designed for either fixed or expansion bearings.

All bearings should be accessible for inspection and maintenance. For bearings that are designed for longitudinal movement, the plans should include, in tabular form, the required settings throughout the probable temperature range at the time of erection or construction.

The designer should keep construction procedures in mind and carefully detail bearing seats. Difficult profile grade geometry and skew effects often will require the use of grout pads under bearings. These grout pads are cast after the abutment or pier seat is complete and allow exact bearing location to be achieved. Because it is unreinforced, the thickness of the grout pad should be limited, and the grout pad should be recessed into the bearing seat.

Together with deck joints, bridge bearings are a source of major structural problems and often are the cause of serious damage to other parts of the structure. Bearings must be engineered and designed to allow free movement and to transmit superstructure loads. Careful analysis should be made of all bridge bearings. A *Standardized* bridge bearing that fits all conditions does not exist.

J. Foundations and Substructures. The *substructure* is that part of the structure that serves to transmit the forces of the superstructure and the forces on the substructure itself onto the foundation.

The *foundation* is that part of a structure that serves to transmit the forces of the structure onto the natural ground.

If a stratum of soil suitable for sustaining a structure is located at a relatively shallow depth, the structure may be supported directly on it by a spread foundation. If the upper strata are too weak, the loads are transferred to more suitable material at greater depth by means of piles or piers.

The design of the structural elements for foundations, substructures and retaining walls shall be in accordance with AASHTO provisions.

Some of the items that are determined by evaluation of site investigations and/or by current practice are as follows:

- Bearing capacities of foundation soils.
- Settlement of foundation soils.
- Ability of piles to transfer load to the ground.
- Lateral earth pressures.
- Lateral earth resistances.

In stability analyses, the factors of safety for overturning and sliding are not specified by AASHTO. Determination of values to be used is based on accepted practice and evaluation of the risk involved.

1. Capacity of Shallow Foundations. A *shallow foundation* is a term applied to footings having a depth-to-base width ratio of 1 or less.

Two things control the capacity of a shallow foundation:

- The ability of the soil to support the loads imposed upon it, known as the bearing capacity of the soil.

- The amount of total or differential settlement that can be tolerated by the structure being considered.

2. Capacity of Deep Foundations. A *pier* is a structural member of steel, concrete or masonry that transfers a load through a poor stratum onto a better one. A *pile* is essentially a slender pier that transfers a load either through its tip onto a firm stratum (point bearing pile) and/or through side friction onto the surrounding soil over some portion of its length (friction pile).

In general, the bearing capacity of a single pile is controlled by the structural strength of the pile and the supporting strength of the soil. The smaller of the two values is used for design.

Piles driven through soft material to point-bearing may be dependent upon the structural strength of the pile for their bearing capacity.

The supporting strength of the soil is the sum of two factors (a and b):

a. The bearing capacity of the area beneath the base.

b. The frictional resistance on the contact surface area for the length of the pile.

For point-bearing piles, factor "a" is of primary significance while for friction piles, factor "b" is of primary significance.

Structural sections of piles are to be designed using the provisions for the material being used and satisfying the minimum requirements specified in AASHTO and this section in foundations.

A pile load test is probably the best method available for determining the bearing capacity of an individual pile. The tests are quite expensive, however, and on small jobs, the cost of their use cannot be justified.

3. Substructure Analysis and Design. In the design procedure, the allowable bearing determinations are performed by the geotechnical engineer prior to completion of the approved layout for final design such as the following:

- *Drilled Shafts* - allowable axial load versus depth for several different diameters is given for design flexibility.

- *Spread Footings* - allowable footing pressure and depth are given. When more refinement is called for, allowable pressures versus footing size are given.

- *Piles* - pile types, sizes, pile tip requirements, estimated tip elevation, and allowable load are given.

These foundation recommendations are presented in a report along with the foundation investigation information.

a. Reinforced Concrete Columns. Since these are the most common substructure elements for transferring superstructure loads to the foundations, discussion of other types will not be included. Reinforced concrete columns are designed by the load factor method for the factored load groups described under section 10.4.B.

Commonly used shapes are round, rectangular, rectangular with rounded ends, and rectangular with large chamfered corners. Flares and tapers are often required. The designer should obtain help from the FLHD bridge engineer and/or the senior structural engineers in determining the type and trial dimensions. The final design should provide adequate strength to cover all factored axial load plus axial or biaxial moment combinations magnified for slenderness as necessary. See section 10.4.E for additional information on reinforced concrete column design. Also, see the seismic specification for extra requirements that ensure connection strength, shear strength, and ductility.

b. Drilled Shafts. The designer usually receives allowable axial load information versus embedment for the foundation type from the geotechnical engineer. In addition to satisfying that the maximum unfactored axial loads do not exceed the allowable, the designer must also perform a lateral-load analysis. The designer must first consider the possible loss of the ground due to scour or erosion. Then shears and moments can be calculated at this level and applied to an embedded pole analysis to determine shears, moments, and deflections of the drilled shaft below the scoured ground level. See section 10.2 for a hand solution of this embedded pole analysis.

c. Spread Footings. The present state-of the-art procedure for these is based on an allowable pressure (working stress) from the geotechnical engineer. The current direction is to replace this with a procedure that will be consistent with load factor design. Until these new methods are developed, footing pressures are calculated from the unfactored loads and the footing is proportioned so that the allowable is not exceeded. The footing is then designed by the service load (working stress) procedure. See section 10.4.E for additional information.

Additional items to be considered in the design of spread footings are as follows:

- Pressure distribution for loads in kern and outside kern.
- Sliding factor of safety.
- Overturning factor of safety.
- Seismic over-strength requirements.

d. Pile Footings. Similar to spread footings pile footings are currently proportioned for a maximum allowable pile load determined by the geotechnical engineer. These pile loads are used for a service load (working stress) design for the footing. This procedure can be replaced with other approved procedures that are compatible with load-factor design.

Additional items to be considered in the design of pile footings are as follows:

- Uplift limitations
- Load distribution to piles
- Lateral-load analysis
- Local punching analysis
- Seismic over-strength requirements

e. Seals. Seals are required for cofferdam construction of foundation portions below water where the waterhead and soil permeability are too great to be controlled by pumping, diversion of water, etc. The need for seals is usually determined during preparation of the preliminary bridge layout. As rough guide is that seals are required for heads of water more than 3 meters depth. The designer calculates the depth of seals for spread footings at 0.43 times the water head at time of construction. A minimum depth of seal should be 600 millimeters. The factor 0.43 is the ratio of the unit weight of water (1000 kg/m^3) to the unit weight of plain concrete (2320 kg/m^3).

For pile footings where uplift resistance of the piles can be counted on, the seal depth may be reduced to 0.25 times the water head.

K. Retaining Wall Design. This section provides guidance in preparing design calculations, plans, and specifications for retaining walls.

Use the Service Load Design method for design of retaining walls. In general, concrete for retaining walls normally is Class A(AE) with a 28-day compressive strength of 20 to 30 megapascals.

Base earth pressures on soil weight are equal to 1920 kg/m^3, the surcharge slope, the coefficient of internal friction, and/or the cohesion of the backfill material. No structure should be designed for less than an equivalent fluid mass of 576 kg/m^3. At the junction of the abutment or abutment wing and retaining wall that is, an equivalent fluid mass of 864 kg/m^3 should be used. This increased loading can normally be reduced to 576 kg/m^3 at a distance from the junction of the abutment and retaining wall equal to the average height of the wall under design. The retaining wall can also be offset from the abutment to equalize deflection instead of designing a short section of wall with increased rigidity.

A minimum live-load surcharge of 600 millimeters of earth should be used.

The resultant of forces should be kept within the middle one-third of the footing for Group I loadings and within the middle one-half of the footing for all other service load conditions.

The designed safety factor and other wall criteria shall be in accordance with Table 10-3.

Table 10-3
Design [1] Safety Factors for Retaining Walls [5]

Components[2]	Safety Factor	
	Range	Typical
Bearing Capacity	2.5 - 3.0	2.5
Overturning	1.5 - 2.0	2.0
Sliding along base[4]	1.5 - 2.0	1.5
Reinforcement member pullout[3]	1.5 - 2.0	1.5
External Slope Stability	1.25 - 1.50	1.5

NOTE:
[1] Expected design life 50/100 years.
[2] Allowable stress not to exceed 0.55 f_y (yield of stress of steel).
[3] At 19 millimeters maximum deflection.
[4] The passive pressure of the earth in the front of the footing may be considered if the earth is more than 1 meter deep on the top of the footing and does not slope downward away from the wall. Do not consider passive pressure if the wall is subject to scour. Sometimes it may be necessary to slope the footing or to provide a key to resist horizontal thrust. The design soil pressure at the toe of the footing shall not exceed the allowable soil bearing capacity.
[5] Deviations from these safety factors shall be approved by the FLHD Structures Engineer.

1. Special Designs. Special wall designs may be required when surcharge conditions are unusual or exceed those values acceptable for standard wall designs. The surcharge conditions, heights, and types of wall in the standard plans cover most of the applications of retaining walls for highway design.

Railroad live load is an example of a severe surcharge. A building is another example of a severe surcharge. Additional load imposed by sign structures and site conditions that do not permit construction of a standard wall will require a special design.

Ordinarily, some decrease in cost may be realized by a special design for a long length of wall having surcharge conditions less severe than those shown in the standard plans. However, for most cases where the use of a standard wall is practical but conservative, the cost of preparing a special design would exceed the savings.

2. Aesthetic Considerations. The type of face treatment for retaining walls is decided on a job-to-job basis according to degree of visual impact. The wall should blend in with its surroundings and complement other structures in the vicinity. Top of walls are usually on smooth flowing curves as seen in elevation.

The profile of the top wall should be designed to be as pleasing as the site conditions permit. All slope changes at the top of the wall should be rounded with vertical curves at least 6 meters long. Small dips in the top of the wall should be eliminated. Sharp dips should be improved by using vertical curves, slopes and steps, or combinations thereof. Side slopes may be flattened or other adjustments made to provide a pleasing wall profile.

Where walls are adjacent to highways, frontage roads, or city streets, special surface texturing, recessed paneling, or provisions for landscaping shall be considered.

3. Footings. For economy and ease of construction of reinforced concrete retaining walls, consider the following criteria for layout of footing steps:

- Distance between steps should be in multiples of standard plywood sizes.

- A minimum number of steps should be used even if a slightly higher wall is necessary. Small steps less than 300 millimeters in height should be avoided unless the distance between steps is 29 meters or more. The maximum height of steps should be held to 1.2 meters. If the footing thickness changes between steps, the bottom of the footing elevation should be adjusted so that the top of the footing remains level.

4. Wall Joints. For cantilevered and gravity walls, Joint spacing should be a maximum 7 meters on centers. For counterfort wall, joint spacing should be a maximum of 10 meters on centers. For tieback walls, joint spacing should be 7 to 10 meters on centers for cast-in-place walls, but for precast units, the length of the unit would depend on the height and thereby the weight of the unit. Odd panels for all type of walls should normally be made up at the ends of the walls. For cast-in-place construction, a minimum of 12-millimeter premolded filler should be specified.

No joints other than construction joints should be used in footings except at bridge abutments and where the change from a pile footing to a spread footing occurs. In these cases, a 12-millimeter premolded expansion joint through the wall and a construction joint with shear keys through the footing should be used. In addition, dowel bars should be placed across the footing joints parallel to the wall elements to guard against differential settlement or deflection of the footings.

The maximum spacing of construction joints in the retaining wall footing should be 36 meters. The footing construction joints should not line up with the expansion joints in the wall.

5. Drainage. Gutters should be used behind walls in areas where there is a necessity to carry off surface water or to prevent scour. Low points in the vertical wall alignment or areas between return walls must be drained by downspouts passing through the walls.

The standard plans show typical drainage details. Special design of surface water drainage facilities may be necessary depending on the amount of surface water anticipated.

Where ground water is likely to occur in any quantity, special provisions must be made to intercept the flow to prevent buildup of hydro-static pressures and unsightly continuous flow through weep holes.

All concrete retaining walls should have 100-millimeter diameter weep holes located 200 millimeters above final ground line and spaced about 4 meters apart. In case the vertical distance between the top of the footing and final ground line in front of the wall is greater than 3 meters, additional weep holes should be provided 200 millimeters above the top of the footing.

Weep holes can get clogged and the water pressure behind the wall may start to increase. In order to keep the water pressure from increasing, it is of utmost importance to have free draining gravel backfill and underdrains.

6. Other. Make provisions to relocate or otherwise accommodate utilities conflicting with the retaining wall. Any modifications of a standard wall to accommodate utilities should be specially designed.

Show all special wall details such as sign bases, utility openings, drainage features, fences, and concrete barriers on the applicable sheet of the wall plans or on a separate plan sheet and include with the wall plans. Cross reference details between the various plan sheets on which they are shown.

10.5 APPROVALS

This section briefly discusses the steps taken by the division bridge staff to acquire client approval of proposed bridge structure type, size, and location for a given project. Steps taken to obtain such approvals must be both timely and contain adequate detail to maintain assigned program schedules.

A. Bridge TS&L. The first step in acquiring client approval of a proposed structure is to develop one or more drawings that depict the bridge type, size, and location for each site.

Data required to develop a bridge site plan is furnished by the highway design/location staff. A site plan shall include the following:

- A plan view showing the horizontal alignment of the roadway and the ground contours of the surrounding area.
- The vertical alignment of the roadway within the limits of the bridge site.
- The roadway typical section(s) to be used at the site.

After determining approximate span lengths and superstructure depths, the bridge opening shall be checked for adequacy.

For stream crossings, a hydraulic analysis shall be made for the site.

For roadway crossings, vertical clearance above the underpass roadway shall be checked. Once the appropriate clearance checks have been made, the profile grade can be adjusted for final TS&L development.

Once developed, the TS&L drawing is then distributed to the client agency for review and approval. Upon receipt of this approval, the structure design and contract plan development can begin.

B. Design Standards and Exceptions. There are many publications available to the design engineer to aid in the development of engineering design calculations for highway structures. Deviations from specific minimum values therein are permissible only after due consideration is given to all project conditions (such as maximum service and safety benefits, type and purpose of improvement, and compatibility with adjacent sections of unimproved roadway).

Exceptions to design standards are to be documented during the TS&L development stage. All responsible agencies should be made aware of each exception, agree to the need for the exception, and be fully aware of any safety and environmental impacts resulting from the deviation.

C. Plans, Specifications, and Estimate. Upon completion of the final plans, specifications, and estimate (PS&E) for a structure, all documents are to be forwarded to the highway design staff for inclusion with the roadway portion of the project.

The plans and specifications should address and adequately describe the design features incorporated into the structure, the construction requirements necessary to facilitate the building of the structure, and an estimate of construction costs of the project.

The estimate should reflect the anticipated cost of the project based on an analysis of previously bid items of work for structures of similar type and construction and geographic location.

Detailed plans for bridges should contain the following drawings and data:

- Site plan.
- Location and log of each foundation sounding or boring.
- Profile of the crossing.
- Typical cross section.
- Sectional drawings, as needed, to detail the structure completely.
- Quantities of materials required.
- Reinforcing bar list and bar bending diagram.
- Design loadings, working stresses, class(es) of concrete, and grade(s) of steel.
- Drainage area and applicable runoff of hydraulic properties.
- Design and construction details not otherwise covered in the specifications.
- References to applicable standard or industry specifications.

10.6 STANDARD FORMAT

A standard format is required in all plans and specifications. Standard formats have been established for drafting plan sheets, writing contract specifications, and establishing contract unit-bid items. Document storage and retrieval procedures for work developed on the Computer Aided Drafting and Design (CADD) System have been developed. See Chapter 9 for additional guidelines.

A. Plans. Standard formats for plan sheet border size, title block, and project identification data can be found detailed in Chapter 9. A majority of this information is also stored on the CADD system for ready use and reproduction. Standards for appropriate line weight, lettering fonts, and commonly used symbols and details can also be found in Chapter 9 or in the CADD user manual.

B. Specifications. There are 3 major types of specifications used in a contract and they are:

- Standard specifications.
- Supplemental specifications.
- Special contract requirements.

See the definition for contract document hierarchy in Chapter 1, Section 1.4.B.

Each Division office maintains a file of special contract requirements that have been developed for addressing unique or specialty work that may be required due to a project's geographical location or special design features that would not be covered in the standard or supplemental specifications.

When it becomes necessary to develop special contract requirements, they shall be written in the same format as the standard specifications.

C. Estimate. An engineer's estimate is developed in the preliminary PS&E stage of plan development based on an average cost per square meter of bridge deck. As the structural design proceeds toward the final PS&E stage, a revised cost estimate is developed based on a unit price analysis for all items of work to be accomplished under the project.

One source of data that can be used for estimating purposes is past contract unit-bid prices for similar type work within the same geographical area. Caution is urged when establishing unit prices from past records.

When estimating, the engineer must consider the current economic environment and be aware of regional cost trends and industry pricing data.

Estimates should be realistic and should be based on a reasonable cost analysis for the work to be accomplished. Unrealistic estimates (either too high or too low) have a detrimental impact on future project planning and programming.

10.7 DIVISION PROCEDURES

Reserved for Federal Lands Highway Division office use in supplementing the policy and guidelines set forth in this chapter with appropriate Division procedures and direction.

LIST OF EXHIBITS

Exhibit

10.1 Design Methods and Strength Requirements by Ashby T. Gibbons, Jr.

Design Methods and Strength Requirements

By: Ashby T. Gibbons, Jr.

Introduction

Existing historical evidence indicates that there was serious, and probably, heated, debate in the first decade of this century between proponents of two concrete design methods similar (for beam design) to those in the revised AASHTO specifications. The committee established to write the first what is now known as the "ACI Building Code" recommended, in 1908, approval of the design provisions of the existing model code of the insurance industry "based on the assumption of a load four times as great as the total working load producing a stress in the steel equal to the elastic limit and a stress in the concrete equal to 1.4 MPa." The recommendation was overturned at the convention of the National Association of Cement Users (now ACI) in favor of working loads, working stresses and straightline theory. The first Joint Committee on Standard Specifications for Concrete and Reinforced Concrete also decided in favor of working stress design in 1909. It seems evident that the deciding factor was not the properties of reinforced concrete, which were surprisingly well known for flexural members, but the desire of engineers to use the familiar theories used for the design of cast iron, wood, and steel. As a result, conditions at working loads were emphasized in design, the real safety of concrete structures became obscured by the approximations and assumptions of design formulae, and designers made countless calculations of stresses which have little relationship to those existing in structures but which year by year became the object of devout belief.

The decision to use allowable stresses and elastic straightline theory would have served very well if concrete actually had a straightline stress-strain relationship and if concrete did not creep significantly more than steel under stress. It was not long before design procedures had to be modified to account for these properties. Column design for concentric loads and for small eccentricities has not followed elastic theory for years; the effect of compression reinforcement in beams was doubled from that indicated by straightline theory; and the re-evaluation of shear and development of reinforcement in recent years has been based on strength capacity with nominal stresses used only as a convenience. Thus, by the 1960's, the latest methods for proportioning concrete members and those of the 1973 AASHTO Specs used unmodified straightline theory only for flexural members without compression reinforcement.

It so happens that the strength of a beam without compression reinforcement is not influenced significantly by concrete strength, and the straightline theory gives reasonably good results for low percentages of reinforcement.

However, the straightline theory grossly underestimates the ability of concrete to resist moment in compression. Consequently, many designs in the past have required compression reinforcement where it was not actually needed to provide the desired strength.

The straightline theory has been notably unsuccessful in predicting column strength because the stress-strain relationship of concrete has a very significant effect when the member is subjected to axial compressive loads. For many years, the strength addition formula has been used for concentrically loaded columns, a modified strength approach has been used for small eccentricities, and elastic-straightline theory used only for large eccentricities. However, the resulting column designs has a widely varying "factor of safety" which ranged from over 4 to nearly one, depending on material strengths, eccentricity and the amount and distribution of the reinforcement. Therefore, concrete column design throughout the AASHTO specifications has been completely revised and is based on strength relationship for both Service Load Design and Load Factor Design.

Analysis

The methods to be used in determining the load effects on concrete bridge members, and some dimensional limitations for specified members, are given in AASHTO Specifications and are to be used for both the Service load Design and the Strength Design methods. Design rules are included for Expansion and Contraction, Stiffness, Modulus of Elasticity, Thermal and Shrinkage Coefficient, Span Length, Computation of Deflection, Composite Concrete Flexural Members. Shoring. T-Girder Construction, Box Girder Construction and Concrete Arch Construction.

The theory of elastic analysis is to be used to determine the moments and forces resulting from loads on statistically indeterminate structures.. This is true for both Service Load and Load Factor Design The principal difference between the two design methods resulting from analysis is that the live load effects are amplified in the load factor method, since the ratio of the live load to dead load is multiplied by 5/3. For example, the points of contraflexure for negative moments will be farther from the supports, and those for positive moments closer to the supports in Strength Design than in Service Load Design.

Design

The specifications not only assure adequate strength. but include provisions to provide acceptable performance at service load levels. There is not always a clear separation between provisions for strength and those for serviceability. For actions other than flexure, the strength and detailing provisions will assure satisfactory performance at service loads. For flexure, special serviceability provisions have been introduced for Strength Design. These serviceability requirements are not necessary when Service Load Design is used since that method is based upon providing acceptable behavior at service loads.

When the Strength Design method is used, members of reinforced concrete structures shall be proportioned for adequate strength using the capacity reduction factors and the factored loads.

Strength. The usable strength for the design of a member, is that calculated in accordance with the specifications—including a capacity reduction factor, ∅. The specifications are based generally on conservatively chosen limiting states of stress, strain, cracking or crushing, and fits to research data for each type of structural action. An understanding of the physical significance of the strengths computed for various actions can only be obtained by review of the background of the provisions.

The concept of the capacity reduction factor, ∅, is (1) to define a level of strength for design which is less than that what could be expected if all dimensions and material properties were those used in computations, (2) to reflect the degree of ductility, toughness, and reliability of the member under the load effects being considered, and (3) to reflect the importance of the member. For example, a lower ∅ is used for columns than for beams because columns generally have less ductility, are more sensitive to variations in concrete strength, and carry larger loaded areas than beams. Furthermore, spiral columns are assigned a higher ∅ than tied columns since they have greater ductility or toughness.

A sliding ∅ factor is permitted for members subjected to bending and small axial loads. The ∅ may be increased from that for compression members to the 0.90 for flexure as the design axial load decreases from a specified value to zero. The upper value of design load below which an increase in ∅ can be made is $0.10\ f'_c A_g$ or P_b, whichever is less.

Note: P_b will be greater than $0.10\ f_c A_g$ and need not be calculated if f_y does not exceed 400 MPa, the reinforcement is symmetrical and $(h- d'-d_s)/h$ is not less than 0. 70. The distance, d_s, is from the centroid of the tension steel to the tension face of the member.

Required Strength. The basic criterion for load factor design may be expressed as follows:

Required strength $\leq$ Calculated usable design strength

All members and all sections of members must be proportioned to meet this criterion.

This required strength is expressed in terms of design loads or their related internal moments and forces. Design loads are defined as load groups multiplied by the appropriate load factors. The loads to be used are described in the specifications.

Design loads are multiplied by appropriate "load factors" to account for increased load effects possibly resulting from such causes as overloads and inaccuracies in analysis. The designer has the choice of multiplying the loads by the load factors before computing the design load effects, or computing the effects from the unfactored loads and multiplying the effects by the load factors. Under the principle of superposition, both procedures yield the same answer.

The factor assigned to each load was influenced by the degree of accuracy to which the load usually can be calculated and the variation which might be expected in the load during the lifetime of the structure. Hence, dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads.

In assigning factors to combinations of loading, some consideration was given to the likelihood of simultaneous occurrence.

Due regard is to be given to sign in determining the effects from combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type.

Consideration must be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined moment and axial load, or shear strength in members with axial load.