OHIO DEPARTMENT OF TRANSPORTATION
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October 21, 2005

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Bridge Standards Engineer

Re: 2005 Fourth Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, January 2004. This package contains the revised pages. The revised pages have been designed to replace the corresponding pages in the book and are numbered accordingly. Revisions, additions, and deletions are marked in the revised pages by the use of one vertical line in the right margin. The header of the revised pages is dated accordingly.

To keep your Manual correct and up-to-date, please replace the appropriate pages in the book with the pages in this package.

To ensure proper printing, make sure your printer is set to print in the 2-sided mode.

The January 2004 edition of the Bridge Design Manual may be downloaded at no cost using the following link: http://www.dot.state.oh.us/se/BDM/BDM2004/bdm2004.htm

Attached is a brief description of each revision.

Summary of Fourth Quarter, 2005 Revisions to the ODOT BDM

BDM Section	Affected Pages	Revision Description
201.2.6	2-7	The Foundation Recommendation requirements for MSE Wall supported abutments have been clarified.
202.2.3	2-11	The Foundation Report requirements for MSE Wall supported abutments, formerly included in BDM Section 204.4 have been moved to this section.
204.4	2-22 through 2-23	The Foundation Report requirements formerly included in this section have been moved to BDM Section 202.2.3.
302.1.2.1	3-9 through 3-10	QC/QA concrete should not be considered for small quantity pay items. This revision provides some guidelines for when to specify QC/QA concrete.
402.2 through 402.3.1.5	4-2 through 4-4.8	The entire Fatigue Analysis and Fatigue Retrofit sections of the BDM have been revised.
611.2	6-30 through 6-30.2	For projects involving SS 898, QC/QA Concrete for Structures, the approach slab concrete note, formerly note [93], has been replaced with both notes [93A] and [93B].

analysis should include the initial construction cost and all major future rehabilitation/maintenance costs, converted to present dollars. Sufficient preliminary design must be performed for an accurate cost estimate. Cost data information may be obtained from "Summary of Contracts Awarded". This publication is available from the Office of Contracts.

When a rehabilitation alternate involves salvaging existing concrete members, cost overruns should be anticipated and included in the cost analysis. See Section 400 of this Manual for additional rehabilitation information.

201.2.6 FOUNDATION RECOMMENDATION

The Structure Type Study shall include a Foundation Recommendation that consists of:

A. General foundation type (i.e. Drilled Shafts, Friction Piles, Bearing Piles or Spread Footings)
B. Typed boring logs
C. Laboratory test results as follows:
   1. Soil: Water content, particle size analysis, liquid limit, plastic limit
   2. Rock: RQD

For the scour evaluation, Section 203.3(D), provide D_{50} values from the particle size analysis.

When the foundation recommendation for the preferred alternative includes MSE wall supported abutments, the Designer shall provide estimates for bearing pressure, allowable bearing capacity and settlement. Refer to Section 204.4 for additional considerations.

When unique subsurface conditions arise, include a brief narrative in the Foundation Recommendation for justification to obtain extra soils borings.

201.2.7 PRELIMINARY MAINTENANCE OF TRAFFIC PLAN

The various components of the bridge stage construction shall match those of the approach roadway, and the nomenclature used to identify the various stages (phases) of construction shall be the same for the roadway and the bridge (Stage 1 and Stage 2 or Phase 1 and Phase 2).

The Preliminary Maintenance of Traffic Plan shall include a transverse section(s) defining all stages of removal and construction. The following information should be provided:

A. The existing superstructure and substructure layout with overall dimensions (field verified) and color photographs.

B. Type of temporary railing or barrier.

C. Proposed temporary lane widths, measured as the clear distance between temporary barriers, shall be shown. A temporary single lane width of 11'-0" [3350 mm] or greater is preferred; 10'-0" [3000 mm] is the minimum allowable. Minimum preferred lateral clearance from edge of lane to barrier is 1'-6" [500 mm] (ODOT's Location and Design Manual Section 502.14) but Section 502.22 of the L & D manual, allows this lateral distance to be amended for specific sites and conditions. The designer should ensure that lane and lateral clearance requirements are evaluated versus effects of phased construction on a bridge structure.

D. Location of cut lines. The existing structure should be evaluated to determine where the cut-line can be made to provide structural adequacy. Cut lines through stone substructures should be carefully evaluated to maintain structural integrity through staged removals. Temporary shoring may be required and should be considered.

E. Temporary modifications to superelevated sections (existing and/or proposed) on the deck and/or shoulder in order to accommodate the traffic from the phase construction.

F. Width of closure pour. When determining the closure pour width (see Section 300 of this Manual), the designer should investigate the economics of using the lap splices versus using mechanical connectors. Any necessary structure modifications should be included in the cost estimate. Lap splices are preferred and recommended. A reduced closure width may cause transition problems in the finishing of the bridge deck surface when bringing the various phases of construction together.

G. Profile grade, alignment, approximate location and width of temporary structures

H. Location of temporary shoring

201.3 UTILITIES

All utilities should be accurately located and identified on the Preliminary Structure Site Plan. A note should state whether they are to remain in place, be relocated or be removed, and for the latter two, by whom.

Utilities should not be placed on bridges whenever possible.

The type of superstructure selected for a site may be dependent upon the number of utilities supported on the bridge. The request to allow utilities on the bridge shall be made through the ODOT District Utilities Coordinator. Refer to the ODOT Utilities Manual. Utilities shall be installed in substantial ducts or enclosures adequate to protect the lines from future bridge repair and maintenance operations. Utilities shall not be placed inside of prestressed concrete box beams. For some specific detail issues with utilities on bridges refer to Section 300 of this Manual.

Prior to 1931 the slab bridge standard drawing required the main reinforcement to be placed perpendicular to the abutments when the skew angle was equal to or greater than 20 degrees. This angle was revised to 25 degrees in 1931, 30 degrees in 1933 and finally 35 degrees in 1946. The standard drawing in 1973 required the main reinforcement to be parallel with the centerline of roadway regardless of skew angle. Existing exposed reinforcing steel may be used to confirm the direction of the reinforcing steel.

If the skew angle of the bridge is equal to or greater than the angles listed above for the year built, a temporary longitudinal bent will have to be designed to support the slab where it is cut or if possible locate the cutline parallel to the reinforcing if sufficient room exists. For example a bridge built in 1938 with a 25 degree skew does not require a bent, however a bridge built in 1928 with a 25 degree skew does require a bent to be designed.

When utilizing semi-integral construction, the stability of the new part-width superstructure is to be considered. There exists the potential of the superstructure to move laterally either from the effects of the traffic using the new deck or the lateral earth pressure against the approach slab. See Standard Bridge Drawing "SEMI-INTEGRAL CONSTRUCTION DETAILS" for more information.

202.2.3 FOUNDATION REPORT

The Bridge Preliminary Design Report shall include a Foundation Report in accordance with the ODOT Specifications for Subsurface Investigations. The Foundation Report shall include:

A. Investigational Findings
B. Analyses and Recommendations
C. Boring Logs and Undisturbed Test Data

Where the scour evaluation has been identified a potential problem, the probable scour depths, calculated in accordance with Section 203.3(D), should be considered in the design of the substructures; the location of the bottom of footings; the minimum tip elevations for piles and drilled shafts; and the frictional capacity of piles and drilled shafts.

The Foundation Report for MSE wall supported abutments shall include calculations for bearing pressure, settlement and bearing capacity. The report shall also include all construction constraints, such as soil improvement methods, that may be required.

Specific design considerations for each foundation type are presented in the following sections.

202.2.3.1 SPREAD FOOTINGS

The use of spread footings shall be based on an assessment of design loads, depth of suitable bearing materials, ease of construction, effects of flooding and scour analysis, liquefaction and swelling potential of the soils and frost depth. Generally the amount of predicted settlement of the spread footing and the tolerable movement of the structure control the type of footing. To establish tolerable movements, engineering judgment should be used (also refer to FHWA's Manual on Tolerable Movements, Report No. FHWA/RD-85/107).

The allowable bearing pressure for the foundation soil is a function of the footing dimensions, depth of overburden and the location of the water table. Procedures for computing allowable bearing pressure for both cohesive and cohesionless soils are given in the FHWA Manual “Soils and Foundations Workshop Manual”, Publication No. FHWA-HI-88-009, July 1993. A relationship between Standard Penetration Test (SPT) value, N, and the soil parameters, angle of internal friction, and cohesive strength, c, is given in tables presented in chapter 6 of the FHWA manual. The cohesive strength of soil is taken as one half of the ultimate strength, q_u.

All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term. See Section 600 for notes and additional guidance.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. Preliminary design loads, the estimated size of the footing and the allowable bearing pressure shall be provided for review with the Foundation Report. This information is to be furnished by the design agency preparing the plans.

During the detail design stage, the actual footing size shall be determined based on the actual design loads. Note that the allowable bearing pressure may need to be adjusted for the actual footing size. A safety factor of three (3) shall be used to determine the allowable bearing pressure.

202.2.3.2 PILE FOUNDATIONS

The type, size and estimated length of the piles for each substructure unit shall be shown on the Final Structure Site Plan. Preliminary pile design loads and approximate pile spacings shall be provided with the Foundation Report. This information will be furnished by the design agency preparing the plans. The estimated pay length(s) for the piling shall be measured from the pile tip to the cutoff elevation in the pile cap and shall be rounded up to the nearest five (5) feet [one meter]. Procedures for computing estimated pay length of the piles are given in the FHWA’s “Design and Construction of Driven Pile Foundations, Vols. 1 & 2”, FHWA-HI-97-013/014. Minimum pile tip elevations for friction designed piles may be required and should be shown on the Final Structure Site Plan.

When installing piles at a batter, the site conditions should be studied to determine if installation is practical. Piles under 15 feet [5 meters] in length should not be battered.

202.2.3.2.a STEEL ‘H’ PILES

When piles are driven to refusal on the bedrock, steel ‘H’ piles are generally used. The commonly used pile sizes are:

accordance with the requirements of Section 204.1, except in laminated bedrock such as interbedded shale and limestone, in which case drilled shaft foundations with sufficient embedment into the bedrock are preferred.

D. A scour evaluation shall be performed for all bridges not founded on scour resistant shale or bedrock. All major rehabilitation work requires a scour evaluation. The scour evaluation may simply consist of determining what the bridge is founded on. For example, on a bridge rehabilitation, noting that the bridge is founded on scour resistant bedrock or deep foundations to bedrock, would constitute the scour evaluation. As a minimum, piles shall be embedded 15 ft [4.5 m] below the streambed elevation.

When evaluating a structure for scour, review all inspection reports for evidence of stream degradation (lowering of stream bed), scour or previous scour countermeasures. For existing footings founded on shale, test probe the shale to determine its resistance to weathering and note the relationship of the bottom of the footing to the stream bed elevation.

When it is necessary to calculate scour depths, they are to be calculated by the equations in HEC-18 (Hydraulic Engineering Circular No. 18, Pub. No. FHWA NHI 01-001), "Evaluating Scour at Bridges". The text of HEC-18 should be read in order to understand scour and river mechanics. The references cited in Chapter 3 of HEC-18 are also helpful in understanding the concepts of scour and river mechanics. Scour depths should be considered in the design of the substructures and the location of the bottom of footings and minimum tip elevations for piles and drilled shafts.

A value of Q500 should be used as the super flood is to be estimated by 1.3 x Q100.

204 SUBSTRUCTURE INFORMATION

204.1 FOOTING ELEVATIONS

Substructure footing elevations should be shown on the Final Structure Site Plan. The top of footing should be a minimum of one foot [0.3 meters] below the finished ground line. The top of footing should be at least one foot [0.3 meters] below the bottom of any adjacent drainage ditch. The bottom of footing shall not be less than four feet [1.2 meters] below and measured normal to the finished groundline.

Due to possible stream meander, pier footings for waterway crossings in the overflow section should not be higher than the footings within the stream unless the channel slopes are well protected against scour. Founding pier footings at or above the flow line elevation is strongly discouraged.

Where footings are founded on bedrock (note that undisturbed shale is bedrock) the minimum depth of the bottom of the footing below the stream bed, D, in feet [meters], shall be as computed by the following:

D = T + 0.50Y

Where:
T = Thickness of footing in feet [meters]
Y = distance from bottom of stream bed to surface of bedrock in feet [meters]

The footing depth from the above formula shall place the footing not less than 3 inches [75 mm] into the bedrock.

204.2 EARTH BENCHES AND SLOPES

A bench at the face of abutment shall not be used. Rehabilitation projects may require special slope considerations.

Spill thru slopes should be 2:1, except where soil analysis or existing slopes dictates flatter slopes. The slope is measured normal to the face of the abutment.

For superelevated bridges over waterways, the intersection of the top of slope with the face of abutment shall be on a level line. For other superelevated structures the top of slope shall generally be made approximately parallel to the bridge seat. For structures over streets and roads having steep grades, the intersection of earth slope and face of abutment may be either level or sloping dependent upon which method fits local conditions and gives the most economical and aesthetically pleasing structure.

The spill-thru slope should intersect the face of abutment a minimum of one foot [300 mm], or as specified in a standard bridge drawing, below the bridge seat for stringer type bridges. For concrete slab and prestressed box beam bridges this distance should be 1’-6” [450 mm].

204.3 ABUTMENT TYPES

Preference should be given to the use of spill-thru type abutments. Generally for stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4’-0” [1200 mm]. For stub abutments on spread footing on soil, the minimum dimension shall be 5’-0” [1525 mm]. For any type of abutment, integral design shall be used where possible, see Section 205.8 for additional information.

Wall type abutments should be used only where site conditions dictate their use.

204.4 ABUTMENTS SUPPORTED ON MSE WALLS

When conditions are appropriate, the designer may consider stub type abutments with piling or spread footings supported on MSE walls. Refer to Sections 201.2.6 and 202.2.3 for the staged review requirements. Consult the Office of Structural Engineering for additional design recommendations.

204.5 PIER TYPES

For highway grade separations, the pier type should generally be cap-and-column piers supported on a minimum of 3 columns. (This requirement may be waived for temporary conditions that require caps supported on less than 3 columns.) Typically the pier cap ends should be cantilevered and have squared ends.

For bridges over railroads generally the pier type should be T-type, wall type or cap and column piers. Preference should be given to T-type piers. Where a cap and column pier is located within 25 feet [7.6 meters] from the centerline of tracks, crash walls will be required.

For waterway bridges the following pier type should be used:

A. Capped pile type piers; generally limited to a maximum height of 20 feet [6 meters]. For heights greater than 15 feet [4.5 meters], the designer should analyze the piles as columns above ground. Scour depths shall be considered.

B. Cap-and-column type piers.

C. Solid wall or T-type piers.

Note that the use of T-type piers, or other pier types with large overhangs, makes the removal of debris at the pier face difficult to perform from the bridge deck. For low stream crossings with debris flow problems and where access to the piers from the stream is limited, T-type piers, or other similar pier types, should not be used.

For unusual conditions, other types may be acceptable. In the design of piers which are readily visible to the public, appearance should be given consideration if it does not add appreciably to the cost of the pier.

204.6 RETAINING WALLS

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Retaining Wall Justification shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. A description of the Retaining Wall Justification is provided in Section 1404 of the ODOT Location and Design Manual, Volume Three. Generally, the justification compares the practicality, constructability and economics of the various types of retailing walls listed below:

A. Cast-in-place reinforced concrete

B. Prestressed concrete
C. Tied-back
D. Adjacent drilled shafts
E. Sheet piling
F. H-piling with lagging
G. Cellular (Block, Bin or Crib)
H. Soil nail
I. Mechanically Stabilized Earth (MSE)

Some of the wall types listed above consist of the following proprietary systems with the type of wall shown in parenthesis:

A. Doublewal
Wall type - Cellular Bin

Contact Information:
Doublewal Corporation
7 West Main St.
Plainville, CT 06062
(860)793-0295

B. Mesa Retaining Wall System
Wall type - MSE Modular Block

Contact Information:
Tensar Earth Technologies, Inc.
5883 Glenridge Drive
Suite 200
Atlanta, Georgia 30328
(404)250-1290

C. Ares Retaining Wall System
Wall type - MSE

Contact Information:
Tensar Earth Technologies, Inc.
5775-B Glenridge Drive.
Suite 450
Atlanta, Georgia 30328
(404)250-1290

D. Hilfiker Retaining Walls
Wall type - MSE

Contact Information:
T & B Structural Systems
637 West Hurst Blvd.
Hurst, Texas 76053
(817) 280-9858

E. Reinforced Earth Walls
Wall type - MSE

Contact Information:
The Reinforced Earth Company
1444 North Farnsworth Ave, Suite 505
Aurora, IL 60505
(630)898-3334

F. Retained Earth Walls
Wall type - MSE

Contact Information:
Foster Geotechnical
Division of L. B. Foster Company
1372 Old Bridge Road
Suite 101
Woodbridge, VA 22192
(703) 499-9818

properly performed and are at locations of minimum stress. Construction joints shall be designed to transfer all loads.

302 SUPERSTRUCTURE

302.1 GENERAL CONCRETE REQUIREMENTS

302.1.1 CONCRETE DESIGN ALLOWABLES

A. Superstructure Concrete - Class S or HP:

1. Load Factor Design..4500 psi [31.0 MPa]
2. Service Load Design....... Unit stress = 0.33 x 4500 psi [31.0 MPa] = 1500 psi [10.3 MPa]

B. Substructure Concrete - Class C or HP:

1. Load Factor Design ....4000 psi [27.5 MPa]
2. Service Load Design......... Unit stress = 0.33 x 4000 psi [27.5 MPa] = 1300 psi [9.2 MPa]

C. Drilled Shaft Concrete - Class S Modified:

1. Load Factor Design..4000 psi [27.5 Mpa]
2. Service Load Design..........Unit stress = 0.33 x 4000 psi [25.7 Mpa] = 1300 psi [9.2 Mpa]

302.1.2 SUPERSTRUCTURE CONCRETE TYPES

302.1.2.1 CLASS S & HP CONCRETE, QC/QA CONCRETE FOR STRUCTURES & CONCRETE WITH WARRANTY

Class S Concrete is the Department's traditional concrete mix design for superstructures.

Class HP (High Performance) Concrete mix designs are intended to give a highly dense, very impermeable concrete resulting in a longer structure life. When Class HP Concrete is specified, the Designer shall include the bid item for Class HP Concrete Test Slab. However, the bid item for Class HP Concrete Testing is no longer required because the Department has acquired sufficient test data since the inception of High Performance Concrete.

QC/QA Concrete for Structures, SS898, is a contractor designed mix that meets minimum requirements for strength, permeability and air content. QC/QA Concrete is divided into three classes: substructure (QSC1), superstructure (QSC2) and project specific (QSC3). The contractor assumes responsibility for quality control sampling and testing. Final payment for in-place concrete includes incentives for concrete meeting or exceeding minimum requirements and disincentives for concrete not meeting minimums. QC/QA concrete should not be considered for pay items with less than 100 yd^3 [75 m^3] of concrete.

Class S Concrete for New Bridge Decks with Warranty, SS893, and Class HP Concrete for New Bridge Decks with Warranty, SS894, are standard Class S and HP mix designs that warrant the concrete for a period of seven years against scaling, spalling and cracking. Remedial measures required during the warranty period are to be performed by the original Contractor.

The mix design, curing and placing requirements for both Class S and HP concretes are defined in the CMS.

302.1.2.2 SELECTION OF CONCRETE FOR BRIDGE STRUCTURES

The following concrete types may be specified for superstructure concrete:

A. Class S Concrete
B. Class HP Concrete
C. Class S Concrete for New Bridge Decks with Warranty
D. Class HP Concrete for New Bridge Decks with Warranty
E. QC/QA Concrete Class QSC2
F. QC/QA Concrete Class QSC3

The following concrete types may be specified for substructure concrete:

A. Class C Concrete
B. Class HP Concrete
C. QC/QA Concrete Class QSC1

Contact the District to confirm the selection of concrete type to be used for a specific structure.

High performance concrete shall not be used as a replacement for the drilled shaft concrete specified in 524.

302.1.3 WEARING SURFACE

302.1.3.1 TYPES

A. 1 inch [25 mm] monolithic concrete - defined as the top one inch [25 mm] of a concrete deck slab. This one inch [25 mm] thickness shall not be considered in the structural design of the deck slab or as part of the composite section.

B. 3 inches [75 mm] asphalt concrete - defined as the minimum asphaltic concrete wearing surface to be used on only non-composite prestressed box beams. The asphalt concrete

SECTION 400 – REHABILITATION & REPAIR

401 GENERAL

The technology of bridge rehabilitation and repair is constantly changing. In addition, many of the defects encountered vary from bridge to bridge requiring individual unique solutions. Consequently, this section of the Manual merely presents an overview of bridge rehabilitation and some of the more common types of repairs. The repairs that are discussed are all proven to be reasonably successful and are approved by FHWA for use on federally funded projects. ODOT's District maintenance teams and the Office of Structural Engineering are continually experimenting with new techniques, many of which appear promising, but have not yet reached a point where conclusions can be drawn with regard to their longevity. Until these products and procedures are evaluated, they will not be included in this Manual and they should not be used on Federal aid projects.

401.1 DESIGN CONSIDERATIONS

For individual members, it will be necessary to determine whether the best option is to repair or replace. In making this decision, cost shall be considered along with factors such as traffic maintenance , convenience to the public, longevity of the structure, whether the rehab is long term or short term, and the practicality of either option.

Due to the variation in the types of problems encountered, the designer shall perform an in depth inspection of the structure to identify the defects that exist, and develop a solution which is unique to the problems found. This field inspection should include color photographs and sketches showing pertinent details and field verified dimensions.

It is imperative that an in depth, hands on, inspection of bridges be made, by the Design Agency preparing the repair or rehab plans, to determine the extent of structural steel and concrete repairs. This inspection shall be made concurrent with plan development. Large quantity and cost overruns result when this inspection is not adequately performed resulting in substantial delays to completion of the project.

All pertinent dimensions that can be physically seen shall be field verified or field measured by the designer and incorporated into the plans. It is not permissible to take dimensions directly from old plans without checking them in the field because deviations from plans are common. Every attempt shall be made to prepare plans that reflect the actual conditions in the field. However, it is recognized that uncertainties may exist. Consequently, the note entitled "EXISTING STRUCTURE VERIFICATION", found in Section 600 of this Manual, should be included in the plans with the understanding that the designer is still responsible for making a conscientious effort to provide accurate information based on field observations.

Sketches of various details have been provided throughout this chapter. These sketches are not complete nor are they to be taken as standard details. They are offered as suggestions or ideas for the designer to use in developing his or her own solutions to the unique problems they encounter.

A bibliography has been included at the end of this section. While these references contain much information and many innovative ideas, designers are advised to discuss untested solutions with the Office of Structural Engineering before completing detail plans.

401.2 STRENGTH ANALYSIS

When analyzing existing superstructures, substructures and foundations for strength, the live load is to be the HS20-44 [MS18] truck (or lane load) and the Alternate Military Loading as defined in Section 3.7 of AASHTO.

In analyzing the strength of existing superstructures, substructures and foundations for bridges that are to receive a new deck, a future wearing surface of 60 psf [2.87 kPa] shall be included in the dead load.

402 STRUCTURAL STEEL

402.1 DAMAGE OR SECTION LOSS

It may be necessary to repair a section of a steel member that has been damaged by rust or other means. Welded repairs are not permitted in tension zones. Damaged sections in tension zones normally shall be repaired by bolting new steel to existing steel. The specifics of the details are left to the ingenuity of the designer due to the vast number of possible solutions. If it is absolutely necessary to perform welded repairs in a tension zone, consult the Office of Structural Engineering for recommendations. The designer will be responsible for describing the welding procedures, non-destructive testing (NDT) requirements, etc. in plan notes.

Welding is permitted in compression zones provided the designer ensures that the chemistry of the existing steel is such that it can be welded. This will require either review of old mill certifications or actual sampling of the material for chemical analysis. The designer shall make this determination. Pay close attention to American Welding Society (AWS) Specifications. Field NDT of the welds will be required and it will be necessary to specify the type and location of the NDT in the plans.

402.2 FATIGUE

402.2.1 GENERAL

Fatigue damage, as it pertains to bridges, is typically categorized as due to either load-induced or distortion-induced (displacement-induced) stresses. While both types of damage are actually load-induced, the former is directly related to the application of live load and the resulting stresses (in plane), while the latter is the result of secondary stresses (out of plane) typically transmitted by a secondary member as it tends to change the shape of, or distort, the primary member because of displacement. Load-induced damage is dependent on stress range, type of detail and the number of applications of live load and can be accounted for during design. Distortion-induced damage is not directly quantified in the design of a bridge but can be minimized through proper detailing. Two conditions are necessary for distortion-induced damage to occur: a periodic out-of-plane force or displacement; and an abrupt local change in stiffness where the force/displacement is applied.

Only load-induced fatigue is directly addressed by this section. Since the repairs for distortion-induced fatigue damage are specifically dependent upon the existing detail itself, the repair details are largely up to the designer; are beyond the scope of this section; and may require a special analysis. The designer should consult with the Office of Structural Engineering when developing these types of repairs.

If cracks can be visually detected, the vast portion of the fatigue life has been exhausted and retrofitting of the detail may be necessary.

When Ohio began using the concept of continuous spans, one commonly used standard detail has now become a fatigue concern. The detail consisted of simple span rolled beams butt welded together at the piers with short plates welded to the top and bottom flanges, roughly centered on the piers. The weld line of the beam webs had coped holes at the top and bottom flange, serving as access holes for the flange welds. The plates are not structural in that they are not serving the part of moment plates but were termed “splice plates” and were meant to reinforce the welded flanges, using a “belt and suspenders” logic. These details are illustrated on retired Standard Bridge Drawing SD-1-63, sheet 1, and were generally used with the rolled beams shown in retired Standard Drawing CSB-1-63. When beams of differing depths were used, the web of the shorter beam was cut horizontally and its flange was raised to match the depth of the adjacent beam and the gap in the web was then filled with weld. This method of joining typically resulted in the top splice plate being kinked or bent at the center of the pier. Henceforth, this detail will be referred to as the “Beam Continuity Weld” throughout Section 402.

402.2.2 FATIGUE EVALUATION

A fatigue evaluation of existing steel members/details to be re-used or rehabilitated is required. The evaluation consists of screening the member/detail and performing a remaining life analysis when necessary.

A fatigue evaluation submission shall be made to the Department, as part of the STRUCTURE TYPE STUDY, for final determination as to whether the members require fatigue related upgrading.

402.2.3 DETAILS TO CONSIDER

Evaluate longitudinal rolled steel beams/welded steel girders having either positive moment or negative moment welded coverplates, transverse floor beams with welded coverplates, and/or Beam Continuity Welds with and without splice plates.

If the details of concern have experienced severe corrosion, more than two heat-straightenings, mechanical damage, previously repaired fatigue damage or are wrought iron instead of steel, a special analysis is necessary and is not covered in this section. Contact the Office of Structural Engineering for guidance.

Do not evaluate base metal of rolled beams, welded plate girders without coverplates, cross-frames, cross-frame connections to beams/girders, transverse web stiffeners, longitudinal web stiffeners, lateral bracing, uncracked web welds, riveted members and riveted connections except if required by the Scope of Services.

A special analysis is necessary to evaluate welded truss members, box girders, curved beams, welded steel pier caps, dog-legged rolled beams, details/members with cracks and load carrying diaphragms of curved structures. Contact the Office of Structural Engineering for guidance.

402.2.4 EVALUATION PROCESS

Use the following retrofitting policy when evaluating the re-use of the existing steel beams/girders for deck replacement projects:

402.2.4.1 COVERPLATES AND BEAM CONTINUITY WELDS WITH SPLICE PLATES

A. All ends of coverplate/splice plate locations shall be retrofitted on bridges, without performing remaining fatigue life calculations, when:

1. The bridge is known to contain fatigue cracks at these locations.

2. The bridge is located on the interstate (urban and rural), rural principal arterial - other, and urban principal arterial - other freeway/expressway routes [functional classifications 1, 2, 11 and 12 respectively]. Information regarding functional classification can be obtained from the Office of Urban and Corridor Planning website.

3. The bridge is located on a route that is identified as a route carrying unusually heavy truck loads (i.e., quarry, mining, logging, landfills, heavy manufacturing, etc.). The District shall identify these routes at the time of the pre-scope.

An economic analysis shall be completed comparing the cost of utilizing the existing steel members with all associated repairs, retrofitting, strengthening, painting and widening issues considered, versus superstructure replacement to justify the decision to retrofit. For this analysis, approximately $ 8.00 per pound (in 2005 dollars) may be used for the estimated cost of "Structural Steel Repair". For an estimation of the required weight of steel required for the retrofit, the plate sizes from the retired Bolted Splice Standard Drawing (Standard Bridge Drawing BS-1-93, sheet 2 of 3) may be used with the resulting weight increased by 25 percent if the web is not to be retrofitted and with no increase if the web is to be retrofit. When using the standard drawing, no deduction for bolt holes is necessary. The cost estimate should be based on initial cost.

B. Consider retrofitting the ends of coverplate/splice plate locations of a bridge, or parallel bridges (left-right) that share a continuous deck, that has a one way present day ADTT (Average Daily Truck Traffic) exceeding 1000 trucks. For bridges meeting this criterion, a fatigue life analysis shall be completed to calculate the remaining fatigue life using the procedures shown herein.

   1. When the remaining fatigue life is equal to or exceeds 50 years, retrofitting is not required.

   2. When the remaining fatigue life of any one location is less than 50 years, both ends of the plate at that location shall be retrofitted. Only retrofit those locations with insufficient remaining fatigue life. An economic analysis shall be completed as detailed earlier.

C. Bridges not meeting criteria (A) or (B) do not require a fatigue analysis. No retrofit is required.

When retrofitting the ends of splice plates that are used in conjunction with the Beam Continuity Weld, the top splice plate shall also be retrofitted at the location of the flange splice weld.

Retrofit web welds only when cracked.

On deck repair projects, including overlays and railing upgrades, retrofitting is not required unless known fatigue cracks exist.

402.2.4.2 BEAM CONTINUITY WELD WITHOUT SPLICE PLATES

The flange and web welds are to be retrofitted only if cracks can be visually detected or if there is physical evidence of cracking such as rust stains, damaged paint, etc. In addition to their own field review of the structure as required by Section 401.1 of this Manual, the Designer shall also review the bridge inspection records (available from the District) to ascertain if cracking was noted during an inspection.

The locations that are to be retrofit are only those where the welds are cracked, or show signs of cracking.

402.2.5 CLASSIFICATION OF DETAILS

The end of the coverplates and splice plates and the region of a splice plate directly above the Beam Continuity Weld shall be classified as either an E or an E' detail. In order to qualify as a Category E detail, all of the following criteria must be met otherwise it is to be classified as a Category E' detail:

A. The thickness of the coverplate must be less than 1.0 inch. **AND**
B. The thickness of the top flange of rolled beam must be less than or equal to 0.8 inch. **AND**
C. The coverplate is narrower than the beam flange (with or without welds across the ends of the plate) or the coverplate is wider than the beam flange with welds across the ends of the plate.

The Beam Continuity Welds are typically classified as a Category B detail. However, due to the typical poor weld quality and the lack of testing, for the purpose of a remaining life analysis, these details shall be evaluated based upon the requirements of a Category C detail.

For the purpose of a remaining life analysis, riveted built-up sections, riveted members and riveted connections shall be evaluated based upon the requirements of a Category C detail.

All other details shall be classified according to AASHTO Standard Specification Table 10.3.1B.

402.2.6 REMAINING LIFE ANALYSIS

The remaining life is to be calculated by dividing the total fatigue life into two periods in which the truck volume and fatigue truck weight remain constant over the individual time period as detailed in AASHTO'S "Guide Specification for Fatigue Evaluation of Existing Steel Bridges", 1990 and all Interims, Section 3.2, Alternate 1:

A. A past period from the opening of the bridge to the present, Yp, based on the existing configuration of the beam,

B. A future period from the present to the end of the fatigue life, Yf, based on the proposed configuration of the beam.

Use MEAN LIFE for redundant main members and SAFE LIFE for non-redundant members which, for the purposes of the remaining fatigue life analysis only, are defined as floor beams and superstructures with three or less main longitudinal load carrying members.

The following modifications are to be made to the procedures outlined in the Guide Specification for the calculation of the remaining life:

A. IMPACT (Guide Specification section 2.4)

The gross weight of the fatigue truck is to be increased by 15 percent to account for impact, unless specific site conditions would require an increase up to a maximum of 30 percent. Examples of site conditions would include a severe change in longitudinal grade either directly on or coming onto the bridge or an unusual bunching of trucks.

B. MEMBER SECTION PROPERTIES (Guide Specification section 2.7)

No increase in section properties is to be used to account for any unanticipated possible composite action except when the deck has/is/will be mechanically connected to the beams by means of shear connectors or other structural shapes and meet the reinforcing requirements of AASHTO Standard Specification section 10.50.2.3. Rivets and bolts used in flange splices/connections are not considered a mechanical connection.

C. AVERAGE DAILY TRUCK VOLUME ESTIMATE (Guide Specification section 3.5)

1. Present Truck Volume, T_P

   The average daily truck volume for the past time period is to be estimated using the current ADTT. If the ADTT in one direction is not available, multiply the total ADTT by 0.55 and use the resulting value.

2. Future Truck Volume, T_N

   The average daily truck volume for the future time period is to be estimated using the design ADTT ("20 years hence"), multiplied by a factor of 1.85. If the ADTT in one direction is not available, multiply the total ADTT by 0.55 and use the resulting value.

D. Fatigue Truck weight ratios, W_P/W and W_N/W, shall be taken as unity (1.0).

402.2.7 WIDENING PROJECTS

If an existing fascia beam is to be used as an interior beam on the new widened deck, the fatigue life analysis is to reflect this (i.e. the estimate of the existing damage for the past time period is to be based on the beam being a fascia beam and the estimate of the future damage for the future time period is to be based on the beam being used as an interior beam). Appropriate distribution factors are to be used.

402.2.8 PERMIT LOADS

In addition to the Interstates, the following routes have a large number of permit loads: US 23 in Franklin, Delaware, Marion, Wyandot, Wood and Lucas counties; US 30 in Van Wert, Putnam and Allen counties; and US 224 in Hancock, Seneca, Huron, Ashland and Medina counties.

In order to account for permit loads, if the bridge under consideration is located on any of the routes listed, the ADTT (present and future) is to be increased by 15 percent prior to performing the evaluation and calculating the remaining life.

402.2.9 VERTICAL CLEARANCES

The retrofit of a bottom flange could potentially reduce the amount of vertical clearance provided below an overhead structure due to the plate or the bolt head and needs to be checked by the designer. If the clearance is reduced and is unacceptable to the District, a modified retrofit will be necessary. The retrofit consists of a combination of a partial bolted plate along with a mechanical treatment of the existing weld. Contact the Office of Structural Engineering for further guidance in developing the details.

402.2.10 FATIGUE SUBMISSION INFORMATION

The fatigue submission shall include the following as a minimum:

A. Recommendations from the consultant regarding whether or not the details require retrofitting and are suitable for retrofitting.

B. Recommendations from the consultant regarding any necessary strengthening of members to meet loading requirements.

C. A summary table showing the following at each detail and location being evaluated: remaining fatigue life; moments; stress ranges; fatigue detail category; live load distribution factor; ADTT, whether ADTT is for one way or total traffic; directional distribution factor; present age of structure in years; impact percentage.

Also include the detailed calculations showing the remaining life computations and the economic analysis.

DO NOT submit voluminous pages of meaningless computer output. Provide the information in summary table form only.

402.2.11 SPECIAL ANALYSIS

For structures where the fatigue evaluation procedure outlined in this section, or portions of it, does not apply, a “special analysis” may be necessary. The “special analysis” might consist of: an evaluation procedure based upon specific research findings (e.g. corrosion); the solving of a two-dimensional or three dimension model, as discussed in Section 301.3 of this Manual, in order to determine distribution factors or member stresses or secondary stresses (distortion); a fracture mechanics based analysis; instrumentation to determine member stresses under normal traffic; or in extreme situations, a load test combined with instrumentation in order to determine distribution factors and member stresses. Since each situation is unique, the Designer shall develop an appropriate approach in order to conduct a comprehensive fatigue evaluation while maintaining close coordination with the District and Office of Structural Engineering for guidance and input. The Designer shall submit the "FATIGUE EVALUATION PROCEDURE" to the District for review and approval. The Designer shall then conduct the evaluation accordingly.

402.3 FATIGUE RETROFIT

402.3.1 RETROFIT DESIGN

402.3.1.1 GENERAL

Details that have cracked or whose remaining fatigue life has been determined to be less than the desired service life are to be retrofitted. The retrofit is done by bolting splice plates at the end of the existing plates and/or over the Beam Continuity Weld in accordance with the procedures outlined in the following sections.

When computing the initial moments for composite steel members, use assumptions that will produce the greatest stiffness at the detail being evaluated (AASHTO Standard Specification 10.38.1.6).

402.3.1.2 END OF COVER PLATES AND SPLICE PLATES (BEAM FLANGES)

A. The design is to be in accordance with the AASHTO Standard Specification for Highway Bridges (current edition and all Interims) and the Bridge Design Manual except as noted in this section.

B. The flange splice plates are to be designed for the entire moment, both from dead load and live load at the section and any contribution from the web is to be discounted, except for when the web is also retrofitted.

C. The connection is to be designed as a slip-critical connection.

D. The live load truck shall be the standard HS20 truck (or lane loading as appropriate) as shown in Figure 3.7.7A (3.7.6B) of the AASHTO Standard Specifications for Highway Bridges. A future wearing surface of 60 pounds per square foot is to be included. Standard AASHTO distribution and impact factors are to be used. The loads are to be calculated at the location of the detail.

E. The stress range in the flange, and in the splice plates, must not exceed the allowable stress range for a Category B detail with over 2,000,000 cycles.

F. The bolted splice designs shown on the retired Standard Bridge Drawing, BS-1-93, are not to be used directly for the design but may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts.

G. For coverplates with tapered ends, the number of bolts required for design shall be placed before the beginning of the taper and beyond the end of the taper. Place additional seal bolts in the range of the coverplate taper.

H. Filler plates equal to the thickness of the coverplate/splice plate shall be used and detailed in the plans, as required.

I. If the flange is cracked and the crack extends into the beam web, a properly installed high strength bolt shall be placed in a hole drilled through the web to eliminate the crack tip. If the web crack is longer than one sixth of the beam depth, the web shall be spliced. The plans shall detail the drilling of the hole and the installation of the bolt.

J. Investigate if vertical clearances are affected when retrofitting bottom flange plates.

K. This design procedure applies equally to coverplate and splice plate ends on the bottom and top flanges.

L. When plating the top flange splice plate for the Beam Continuity Weld detail, the designer may be tempted to use one long continuous plate in lieu of three smaller individual plates. However, because of the possible bend in the splice plate, bolting the new plate into position would be difficult. A single plate could be locally yielded when forced into position and the resulting connection would most likely not be slip-critical. Three individual splice plates shall be used.

402.3.1.3 BEAM WEBS

A. The design is to be in accordance with the Standard Specification for Highway Bridges and the Bridge Design Manual except as noted in this section.

B. When the web retrofit is located under an end of a coverplate retrofit, the web splice plates are to be designed for the applied shear, from both dead load and live load. All of the moment is assumed to be taken by the flange. The plates are to be centered around the ends of the coverplate.

   If the web retrofit is located at a Beam Continuity Weld, such as over a pier, and the beam flange is plated, the web splice plates are to be designed for the entire shear, the portion of the moment that is carried by the web and the moment caused by the eccentricity of the bolts.

C. The connection is to be designed as a slip-critical connection.

D. The live load truck shall be the standard HS20 truck (or lane loading as appropriate) as shown in Figure 3.7.7A (3.7.6B) of the AASHTO Standard Specifications for Highway Bridges. A future wearing surface of 60 pounds per square foot is to be included. Standard

AASHTO distribution and impact factors are to be used. The loads are to be calculated at the location of the detail.

E. The stress range in the retrofitted web, and in the splice plates, must not exceed the allowable stress range for a Category B detail with over 2,000,000 cycles.

F. The bolted splice designs shown on the retired Standard Bridge Drawing, BS-1-93, are not to be used directly for the design but may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts.

G. Filler plates equal to the difference in thickness of the beam webs shall be used and detailed in the plans, as required.

H. The crack is to be arrested by drilling a hole at the crack tip.

402.3.1.4 FILLER PLATES

Filler plates shall be considered underdeveloped and their effects shall be considered in the design of the fasteners (AASHTO Standard Specification 10.18.1.2). The design reduction factor shall be applied to the fasteners on both sides of the connection to ensure a symmetrical splice. Filler plates need not extend beyond the plates.

The minimum thickness for a filler plate shall be 1/8 inch, which inherently means that when the difference in thickness of the two sections being plated is less than 1/8 inch, no filler plate is necessary.

When plating the top flange splice plate for a Beam Continuity Weld detail, the designer will need to determine the thickness of the filler plate by examining the details shown on sheet 1 of retired Standard Bridge Drawing SD-1-63. A copy of the drawing can be obtained by contacting the Bridge Standards Engineer in the Office of Structural Engineering.

Filler plates shall be detailed in the plans.

402.3.1.5 FATIGUE CRACKS IN OTHER MEMBERS AND DETAILS

Since the repair of fatigue cracks in members and details is dependent on the specifics of a situation, it is not practical to provide specific guidelines or repair details, so only a general discussion will be made. In order to properly devise a repair scheme, it must be determined if the crack resulted from load-induced or distortion-induced stresses. The source of the cracking should also be identified. More than one factor could contribute to cracking. Defects from poor fabrication may need to be considered. Cracking can also occur during shipment and/or erection. The retrofit may need to include modifying the stiffness of an existing connection. The repair should be logically thought out and its influence on the other steel members investigated. A poorly designed repair may actually worsen the situation as repaired cracks can reinitiate and propagate further into the member, the repair detail itself may experience cracking and new cracking may occur elsewhere if load patterns were modified or member response to loads has been modified.

402.3.2 BOX GIRDER PIER CAPS

Often box girders were constructed using non-continuous back-up bars that were stitch welded in place. The discontinuity in the back-up bar is of major concern since it acts like a crack in the member and is the source of crack propagation into the flange or web. One possible solution is to drill a horizontal hole through the web and back-up bar at the points of discontinuity. See Figure 401 for a sample detail. The stitch welds may or may not be a problem depending on the stress ranges at their location.

402.3.3 MISCELLANEOUS FATIGUE RETROFITS

Various retrofits have been used for fatigue prone details such as small web gaps which result in stress concentration and subsequent cracking, intersecting welds, lateral connection plates, longitudinal stiffeners, cracks and many others. The Office of Structural Engineering may be contacted for assistance in determining the best retrofit for specific details.

402.4 STRENGTHENING OF STRUCTURAL STEEL MEMBERS

Welded stud shear connectors shall be installed full length on all steel beam or girder bridges in which the deck is being removed and replaced. The stud spacing shall be designed in accordance with AASHTO Section 10.38.5.

Bolted cover plates in tension zones or field welded cover plates in compression zones can be used to increase strength. Field welding is to be performed in strict compliance with AWS Specifications. Field NDT of the welds will be required and it will be necessary to specify the type and location of testing in the plans. Also, practicality of field welding shall be evaluated. Overhead welding is not practical.

Consider jacking the stringers to relieve stresses prior to installing cover plates. In this manner the cover plates will carry dead load <u>and</u> live load stresses. If the plates are installed without relieving the stresses, they will carry live load only. This is merely a suggestion as to how extra strength might be obtained if it is needed.

Other methods of increasing the strength are to attach angles or structural shapes to the web or flanges. The possibilities are numerous and shall be left to the ingenuity of the designer. However, the designer shall remember to pay strict attention to practicality as well as strength and fatigue requirements. Consult the Office of Structural Engineering for recommendations or to review unusual details with the Office of Structural Engineering before proceeding.

When retrofitting or repairing truss members, the designer shall remember to provide for

610.7 RAILING

Use the following note where the existing parapet is to be refaced. Modify the note accordingly for each specific project.

[48] ITEM 517 - RAILING FACED, AS PER PLAN

DESCRIPTION: This work consists of facing curb style parapets, using cast in place concrete, to obtain the deflector shape as shown in the plans.

REMOVAL: Carefully remove the existing aluminum railing, posts, curb plates, existing concrete curb and bulb angle gutter. Remove all loose or unsound concrete. Remove sound concrete, as necessary, to obtain a minimum 4 inch [100 mm] thickness of new concrete.

NOTE TO DESIGNER: Modify the list of items in the above removal portion of this note as necessary to fit the actual conditions of your particular project.

DOWEL HOLES AND REINFORCING STEEL: Drill dowel holes where shown in the plans. Install reinforcing steel according to Item 510 using epoxy grout, 705.20. Prior to drilling dowel holes, locate all existing reinforcing steel bars in the area of the hole with the aid of a reinforcing steel bar locator (pachometer). If an existing bar is encountered at the same location as a proposed dowel hole, move the dowel hole to either side of the existing bar. The Department will pay for all reinforcing steel, dowel holes and grouting with Item 517.

SURFACE PREPARATION: Thoroughly clean the parapet surface in contact with the refacing with detergent to remove surface contaminants. After detergent cleaning and within 24 hours of placing concrete, blast clean and air broom or power sweep all surfaces in contact with the refacing to remove all spalls, laitance, and other contaminants detrimental to the achievement of an adequate bond. Acceptable blast cleaning methods are high-pressure water blasting with or without abrasives in water, abrasive blasting with containment or vacuum abrasive blasting. Use hand tools as necessary to remove scale from any exposed reinforcing steel. Materials: Concrete shall be Class __*__ (S or HP) with a compressive strength of 4500 psi [31 MPa]. Furnish reinforcing steel according to 709.00, grade 60 [420], with a minimum yield strength of 60,000 psi [420 MPa].

CONTROL JOINTS: Sawcut 1 1/4 inch [32 mm] deep control joints along the perimeter of the parapet as soon as the saw can be operated without damaging the concrete. Place the joint saw cuts at the same location as the existing deflection joints. Use an edge guide, fence or jig to ensure that the cut joint is straight, true and aligned on all faces of the parapet. The joint width shall be the width of the saw blade, a nominal width of 1/4 inch [6 mm]. Seal the perimeter of the control joint to a minimum depth of one inch [25 mm] with a polyurethane or polymeric material conforming to ASTM C920, Type S. Leave the bottom one-half inch [12 mm] of both the inside and outside faces of the parapet unsealed to allow any water which may enter the joint to escape.

METHOD OF MEASUREMENT: The Department will measure this item in feet by the actual length of railing faced between the ends of the existing concrete parapet.

BASIS OF PAYMENT: Payment for this item includes all costs of removal, dowel holes, reinforcing steel, concrete, shrinkage control joints, epoxy injection and inspection platforms. The Department will pay for accepted quantities at the contract price for Item 517, Railing Faced, As Per Plan.

NOTE TO DESIGNER: Include the reinforcing steel in the bar list with appropriate bending diagrams, as necessary, even though the reinforcing steel is included with item 517 for payment. Modify the method of measurement and items of work included in this pay item as necessary to fit your specific project.

611 MISCELLANEOUS GENERAL NOTES

611.1 DOWEL HOLES

[49] Note Retired - See appendix

611.2 APPROACH SLABS

[50] Note retired - see appendix

[50A] Note Retired - See appendix

Item 526, Reinforced Concrete Approach Slabs was developed such that the concrete used in the superstructure would also be used for the approach slabs. The new supplemental specification for QC/QA concrete is not included in Item 526.

Provide both of the following notes on projects that specify SS898, QC/QA Concrete for Structures:

[93A] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (APPROACH SLAB), AS PER PLAN
Furnish approach slabs conforming to CMS 526 except concrete shall be in accordance with Supplemental Specification 898, QC/QA Concrete, Class QSC2. The accepted quantities shall include: concrete, curbs, reinforcing steel, joint fillers, joint sealers, joint seals, and waterproofing. The Department will measure approach slabs by the number of square yards. The Department will initially pay the full bid price to the Contractor upon

completing the work. The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.

[93B] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (DECK), AS PER PLAN

The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.

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