ASCE-INDOT
STRUCTURAL SUBCOMMITTEE
MEETING NO. 39 MINUTES
February 7, 2008

The meeting was called to order at 9:05 am by Mike Wenning. Those in attendance were:

Anne Rearick	INDOT, Structural Services
Naveed Burki	INDOT, Structural Services
Tony Uremovich	INDOT, Structural Services
Ron McCaslin	INDOT, Structural Services
Bill Dittrich	INDOT, Program Development
Mike McCool	Beam Longest & Neff, LLC.
Mike Wenning	American Structurepoint, Inc.
Burleigh Law	HNTB Corp.
Jason Yeager	Gohman Asphalt Company
Dick O'Connor	RQAW Corporation
Michael Matel	Butler, Fairman and Seufert, Inc.

In addition to the attendees, these minutes will be sent to the following:

Greg Klevitsky	INDOT, Structural Services
George Snyder	INDOT, Structural Services
Jim Reilman	INDOT, Construction Management
Tony Zander	INDOT, Materials and Tests Division
Keith Hoernschmeyer	Federal Highway Administration
Mike Obergfell	USI Consultants, Inc.
Steve Weintraut	Butler, Fairman and Seufert, Inc.

A meeting agenda had previously been distributed and the following items were discussed:

1. The December 13, 2007, meeting minutes were approved as written, and have been placed on the INDOT website.

2. The group decided to add Troy Jessup of R. W. Armstrong and Don Bosse of Prestress Services as new members to the group. Minutes of this meeting as well as the next agenda will be forwarded to them.

3. The specification for the adhesive material, which attaches the joint material to the back of the semi integral end bent, will be submitted for approval to the INDOT Standards Committee at their March 2008 meeting. When this specification is approved, the semi integral end bent details will be incorporated into the Design Manual.

4. A design calculation, which specifies the amount of reinforcing steel required in the hammerhead portion of the pier due to the torsion requirements specified in the LRFD Code, has been forwarded to INDOT for their review. The group felt that the torsion requirements should not control the design of the reinforcing steel in this substructure element, and was asking INDOT to investigate this requirement further. Anne Rearick will let the group know if INDOT will revise or waive this requirement. Anne will also contact Leap Software to discuss this item.

5. Mike McCool presented some proposed modifications to the Indiana Design Manual with regards to the design of reinforced concrete decks on beam bridges (Attachment No. 1). Mike proposed using a minimum of #4 bars spaced at a maximum of 8 inches for both mats of steel as shown in his details. With regards to crack control, it was felt that the spacing of the reinforcing steel was more critical than the steel provided. INDOT will review these modifications and report back to the group with its comments.

6. The group reviewed the pavement ledge detail that was presented by Tony Uremovich (Attachment No. 2). The group offered various comments that Tony will incorporate into the detail. It was pointed out that currently the bridge approach slabs are not allowed to be poured continuously with the concrete bridge deck due to INDOT Construction Memo 07-23. The group felt that this pavement ledge detail should be tried out on an upcoming bridge project. It was felt that the designer could specify this detail as optional on the plans. It was also brought up that a special provision could be written to allow the use of this detail. It was also revealed that there is consideration in revising the end bent width to 3'-0" minimum and increasing the pavement ledge width to 9 inches. Tony will make the revisions to the pavement ledge detail and distribute it to the group for review.

7. Anne Rearick reported that INDOT Materials and Tests is extremely close to implementing LRFD for foundation designs.

8. Tony Uremovich passed out a draft memorandum for the precast, prestressed, concrete Hybrid Bulb-T beams (Attachment No. 3). Tony mentioned that INDOT is in the process of incorporating the 48 inch and 54 inch deep beams into the Design Manual. Since these beams are relatively new, there is a very limited amount of cost history on this item. It was pointed out that for the long bulb-T beams as well as these Hybrid beams, contractors are renting hydraulic cranes, which are relatively expensive, to place these beams in place.

9. The concrete deck overhang design for precast concrete bulb-T and Hybrid beams was discussed. Due to the wide top flanges of these types of beams, not much room is available along the outside edge of the exterior beams for drainage on the bridge deck. The contractors prefer to have the coping overhang bracket angles be somewhat steep. It was felt that the current design criteria for the slab overhang distance for these types of beams needed to be looked at. Burleigh Law volunteered to calculate the design loads to the exterior beams at the coping overhangs in accordance with the LRFD code requirements (Attachment No. 4), and report back to the group with his results.

10. Anne Rearick has discussed LRFD training for designers with Erikkson Technologies. The group felt that the training should concentrate on the application of the codes with design examples with a limited amount of theory. It was felt that the individual topics such as concrete, steel, foundations and loading should be covered. Anne will continue to talk with Erikkson and report back on her progress.

11. It was announced that INDOT will be offering a training session on Seismic Design on April 16th and 17th. It will be a one day seminar.

12. It was reported that three bridge projects had problems with respect to the residual camber of the precast, prestressed concrete beams during construction. It was found that after the beams had been set, the actual fillet depths were deeper than what was specified on the plans. It was asked that the bridge inspectors pay special attention to this item this summer to determine if this is a common problem. Anne Rearick will look into this further and obtain more information.

13. Bill Dittrich requested that an inspection manual be produced to aid bridge inspectors for post tensioned bridge inspections. Bill mentioned that the bridge inspectors are not very well trained for this type of bridge inspection.

14. Section 3.6.5.2 and Table A13.2-1 of the LRFD code were brought up for discussion (Attachment No. 5). It was pointed out that when a column is designed for the 400 kip train collision load, the reinforcing steel required was extremely high. It was also pointed out that the LRFD code required substructure elements to be protected by a crashwall if they were located within 50 feet of the centerline of the railroad.

15. At the next meeting, the topic of "Revisions to the Design Manual which are required by the 4th Edition of the LRFD Code" should be discussed.

The next meeting for the INDOT Structural Subcommittee is scheduled for 9:00 am on April 24, 2008, in a room to be determined.

This meeting was adjourned at 11:15 a.m.

Respectfully submitted,
BUTLER, FAIRMAN and SEUFERT, INC.

Michael Matel, P.E.
mmatel@bfsengr.com

MM:lm

Attachments

ATTACHMENT No. 1

Due to the recent changes in the AASHTO LRFD crack control provisions and the national confusion on the accuracy of these provisions, I would recommend the following revision to the IDM. IDM Sections **61-2.02(05) Crack Control** and **61-2.02(06) Minimum Reinforcing Steel** shall not be used for reinforced concrete decks on beam bridges. The following IDM sections could be revised as shown. REV.

61-3.03 Criteria for Empirical Design

The complexity and sophistication that may be required in the computations when dealing with in-plane forces in the inelastic phase is deemed to be beyond the normal scope of design. Instead, the LRFD Specifications provides a set of criteria that must be satisfied if the empirical design is applied. The criteria are repeated herein with commentary added as appropriate.

a. Cross-frames or diaphragms are used throughout the cross section at lines of support.

b. For cross sections involving torsionally stiff units, such as individual separated box beams, either intermediate diaphragms between the boxes are provided at a spacing not to exceed 25 ft, or the need for supplemental reinforcement over the webs to accommodate transverse bending between the box units is investigated and reinforcement is provided if necessary.

c. The supporting components are made of steel and/or concrete.

d. Deck is fully cast in place and water cured. The intent of this requirement is to exclude a deck in which either the reinforcing steel or the concrete, or both, are discontinuous.

e. The deck is of uniform depth, except for haunches at girder flanges and other local thickening. This requirement reflects that all research work was carried out on slabs of uniform depth.

f. The ratio of effective length to design depth does not exceed 18.0 and is not less than 6.0. This is perhaps the most important requirement, by which flatness of the internal arch is limited. Figure 61-3B interprets the effective length of deck S for various support conditions such as AASHTO I-beams, box beams, steel I-beams, or concrete bulb-tee beams.

g. Core depth of the slab is not less than 4 in. The intent of this requirement is to provide adequate internal moment arm for the slab.

h. The effective length, as specified in Article 9.7.2.3, does not exceed 13.5 ft. This requirement reflects upon the maximum size of specimens tested. If effective length, S, exceeds 11 ft, the depth of slab needs to be increased according to Item 6 above. If S is less than 3.5 ft, the strip method of design shall be used. ← Rev.

i. The minimum depth of slab is not less than 7 in, excluding a sacrificial wearing surface. The minimum depth of slab is 8 in, which includes a ½ in sacrificial wearing surface.

j. There is an overhang beyond the centerline of the outside girder of at least five times the depth of the slab; this condition is satisfied if the overhang is at least three times the depth of the slab, and a structurally continuous concrete barrier is made composite with the overhang. The intent is to provide a tension ring of sufficient width at the edge to resist internal arching forces between the exterior and the first interior beam. The concrete barrier railing shown on the INDOT Standard Drawings is considered a structurally continuous concrete barrier.

k. The specified 28-day strength of the deck concrete is not less than 4 ksi. Tests have indicated insensitivity of the deck to compressive strength. The intent is to provide a reasonably resilient and non-permeable deck.

l. Deck is made composite with the supporting structural components. Tests have indicated a definite enhancement of lateral confinement due to composite action.

m. A minimum of two shear connectors at 2 ft centers shall be provided in the negative moment region of a continuous steel beam/girder superstructure. The requirements of LRFD Article 6.10.1.1 shall also be satisfied. For prestressed concrete beams, the use of stirrups extending into the deck shall be taken as sufficient to satisfy this requirement.

n. Stay-in-place concrete formwork shall not be permitted in conjunction with empirical design. A special provision is required for deleting the option of allowing the use of precast deck panels.

The LRFD Specifications requires four layers of isotropic reinforcement. For each of the two top layers, the minimum steel area is 0.18 in^2/ft, and for each of the bottom two layers, the minimum steel area is 0.27 in^2/ft. The recommended minimum reinforcing bars sizes and spacing's for constructability and crack control are as follows:

1. Two top layers, each #4 at 8 in ← REV
2. Two bottom layers, each #4 at 8 in ← REV

Figure 61-3C presents the typical deck reinforcement for the empirical design.

All reinforcement shall be straight bars except where hooks are required. The additional longitudinal reinforcement provided in the deck in the negative moment regions of a continuous beam or girder type bridge, beyond that required for isotropic reinforcement according to the provisions of LRFD Article 9.7.2.5, need not be matched in the transverse direction.

A skewed deck tends to develop torsional cracks in the end zones of the deck. To control crack size, Article 9.7.2.5 of the LRFD Specifications specifies that the minimum reinforcement be doubled in the end zones of the deck, but not at intermediate piers, with a skew in excess of 25°. As shown in Figure 61-3D, end zones, as bounded by dotted lines, are defined for the additional transverse and longitudinal steel. The additional steel should be present at both ends.

In Section 61-3.02, the role of arching effects is discussed and the significance of tightness is stated. The question of tightness (i.e., effective confinement of the compressive zone) is relative to cracking due to shrinkage and/or negative moments in the supporting beams where the slab is in tension. The entire superstructure should be designed and constructed with the objective of minimizing cracking in the deck.

61-4.0 DESIGN DETAILS FOR BRIDGE DECK

61-4.01 General

The following general criteria shall apply to bridge deck design.

a. Thickness. The depth of a reinforced concrete deck shall not be less than 8 in.

b. Reinforcement. The bottom reinforcement cover shall be 1 in. The top reinforcement cover shall be 2 ½ in. The primary reinforcement shall be on the outside. The maximum spacing shall be 8 in on centers for bottom and top reinforcement in both directions. ← REV.

c. Sacrificial Wearing Surface. The top ½ in of the bridge deck shall be considered sacrificial and shall not be included in the structural design or as part of the composite section.

d. Class of Concrete. Class C concrete shall be used.

e. Concrete Strength. The specified 28-day compressive strength of concrete shall not be less than 4 ksi.

f. Reinforcement Steel Strength. The specified yield strength of reinforcing steel shall not be less than 60 ksi.

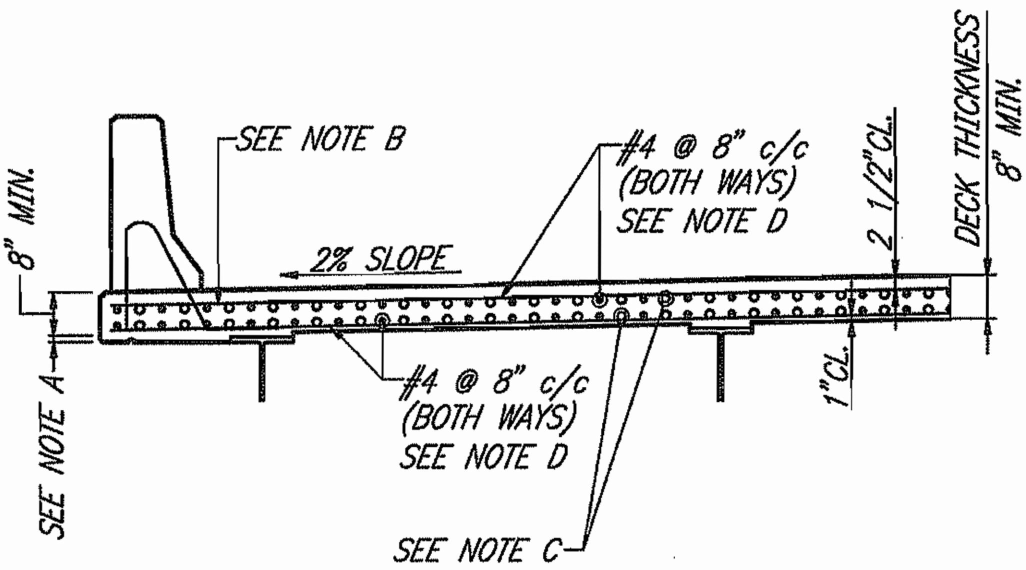
g. Epoxy Coating. All reinforcing steel in both top and bottom layers shall be epoxy coated for a bridge deck supported on beams.

h. Sealing. All exposed roadway surfaces, concrete railings, and outside copings shall be sealed from drip bead to drip bead. In addition, the underside of the copings and the exterior face of outside concrete beams shall also be sealed.

i. Length of Reinforcement Steel. The maximum length of individual reinforcing steel bars shall be 40 ft. All reinforcing bar splice lengths shall be shown on the plans.

j. Truss Bars. Truss bars shall not be used in a concrete deck supported on longitudinal stringers or beams.

k. Placement of Reinforcing Steel. For a skew greater than 25°, transverse reinforcing steel shall be placed perpendicular to the beams. For a skew of 25° or less, reinforcement shall be placed parallel to the skew.



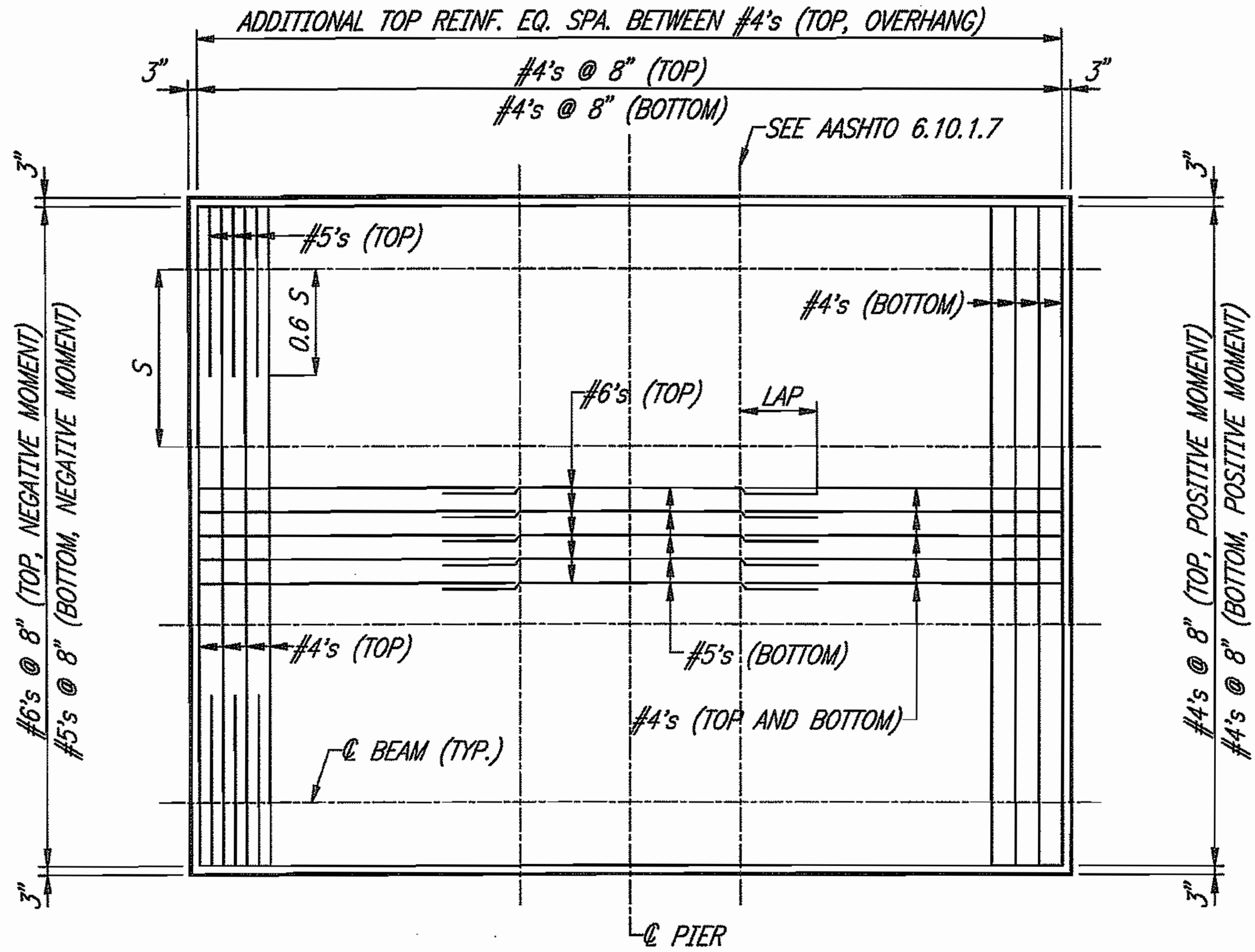
NOTE A: BOTTOM OF DECK FROM UNDERSIDE OF BOTTOM FLANGE TO COPING SHOULD BE SLOPED AS NEEDED OR MADE LEVEL TO MAINTAIN A MINIMUM COPING DEPTH EQUAL TO THE DECK THICKNESS ON TANGENT CROSS SECTIONS.

NOTE B: ADDITIONAL TOP REINFORCEMENT IN DECK OVERHANG REQUIRED BY DESIGN TO BE PLACED EQUALLY BETWEEN TOP #4 BARS.

NOTE C: ADDITIONAL REINFORCEMENT IN THE LONGITUDINAL DIRECTION MAY BE REQUIRED BY DESIGN IN THE NEGATIVE MOMENT REGIONS FOR CONCRETE BEAM STRUCTURES. THE ADDITIONAL STEEL REQUIRED MAY BE PLACED @ 8" CENTERS BETWEEN THE LONGITUDINAL BARS REQUIRED FOR THE EMPIRICAL DECK DESIGN.

NOTE D: REINFORCING SHOWN FOR POSITIVE MOMENT REGION. FOR STEEL STRUCTURES THE TOP LONGITUDINAL REINFORCING SHALL BE #6 BARS @ 8" CENTERS AND THE BOTTOM LONGITUDINAL REINFORCING SHALL BE #5 BARS @ 8" CENTERS PER AASHTO ARTICLE 6.10.1.7.

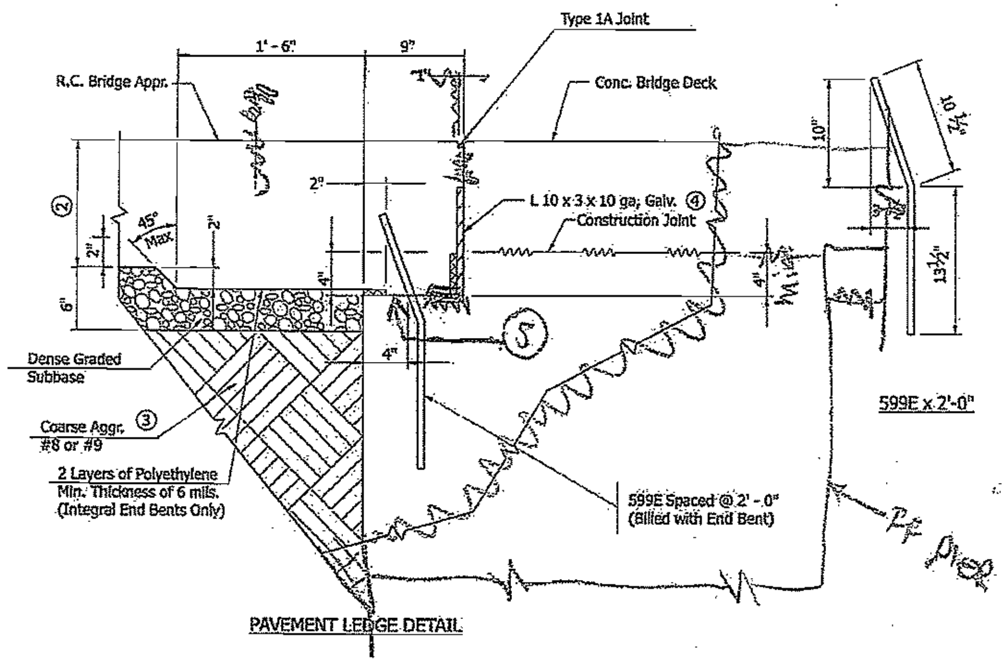
EMPIRICAL DESIGN
(Typical Deck Reinforcement)
Figure 61-3C-1



NOTE:
REINFORCEMENT SHOWN FOR STEEL BEAM OR GIRDER BRIDGES. ADDITIONAL REINFORCEMENT WILL BE REQUIRED FOR CONCRETE BEAM BRIDGES IN THE NEGATIVE MOMENT REGION.

EMPIRICAL DESIGN
(Typical Deck Reinforcement)
Figure 61-3C-2

Attachment No. 2



NOTES:

1. See Standard Drawing E-609-BRJT-01 for Type 1A joint details.

② 10" if design year AADT< 1000
12" if design year AADT ≥ 1000
or match thickness of concrete approach pavement if thicker than 12"

③ Flowable backfill if slab bridge.

④ Angle required only if pouring bridge deck and RCBA simultaneously. Form oil shall be applied to the bridge-deck face of the longer edge of the angle.

LEGEND

⊠ 1/2" Expanded Polystyrene

⑤ The pavement ledge shall be level if the pavement crown is ≤ 2 in. It shall follow the bent's transverse profile if the pavement crown is > 2 in.

INDIANA DEPARTMENT OF TRANSPORTATION	
~~REINFORCED CONCRETE BRIDGE APPROACH PAVEMENT LEDGE DETAIL~~	
~~SEPTEMBER 2007~~	
STANDARD DRAWING NO. E 609-RCBA-07	
	DESIGN STANDARDS ENGINEER — DATE
	CHIEF HIGHWAY ENGINEER — DATE
DESIGN STANDARDS ENGINEER	

MTG 39, AGND ITEM 9
ATTACHMENT No. 3

INDIANA DEPARTMENT OF TRANSPORTATION

Driving Indiana's Economic Growth

Design Memorandum No. 08-__
Technical Advisory

February 6, 2008 DRAFT

TO: **All Design, Operations, and District Personnel, and Consultants**

FROM: ______________________

Anthony L. Uremovich
Design Resources Engineer
Production Management Division

SUBJECT: **Hybrid Bulb-Tee Structural Members**

REVISES: ***Indiana Design Manual* Section 63-4.03**

EFFECTIVE: **______________, 2008, Letting**

New prestresssed-concrete bulb-tee members, identified as hybrid bulb-tees, have been approved for use. One of these sections should be considered if deemed to be the most economical or structurally adequate.

Details and section properties are shown in new *Indiana Design Manual* Figures 63-14Y(1) through 63-14Y(5). Both english- and metric-units versions are attached hereto. The metric-units versions are also posted on the Department's *Indiana Design Manual* webpage.

alu
Attachment

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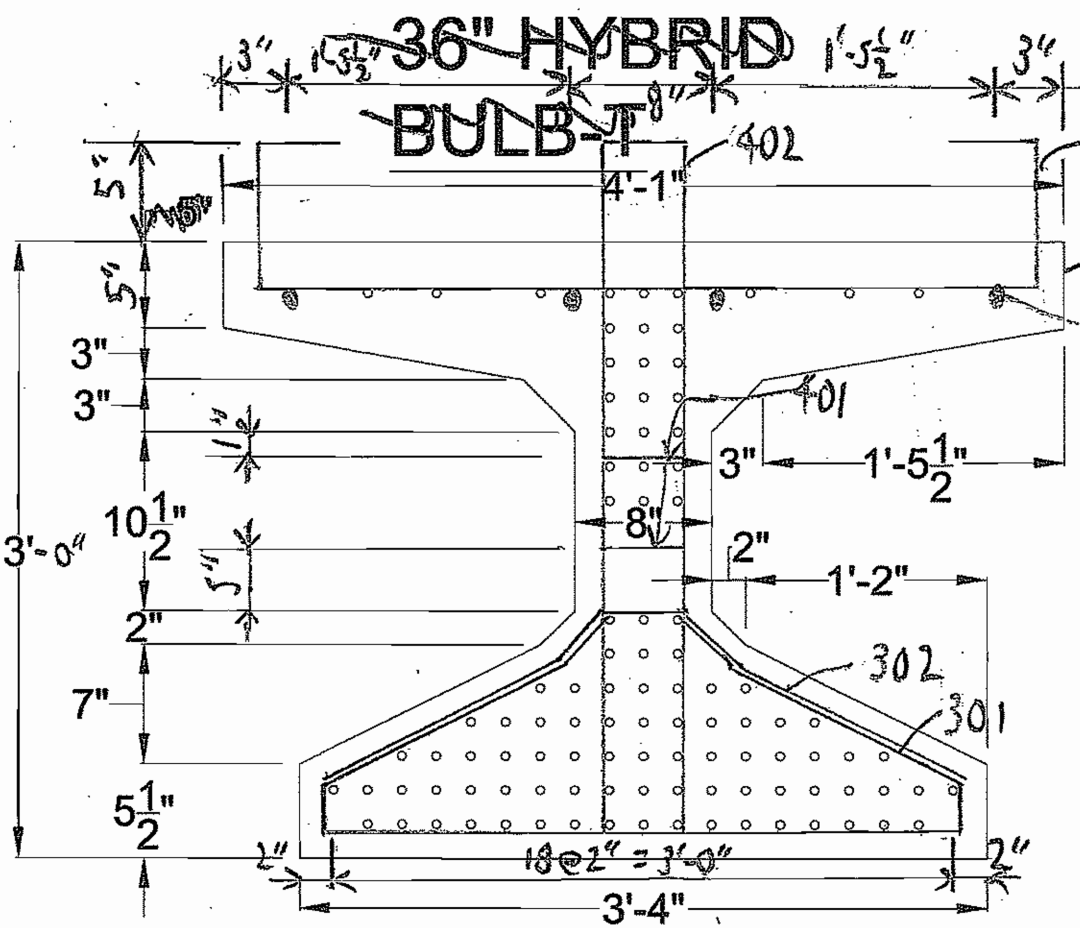
MARKUPS BACK TO ES
M 12-17

~~36" HYBRID BULB-T~~



STRL SUBCOMM MTG 38 ITEM 9

LAYER ENG. DIMENS.

SHOW VERTICAL DIMENSIONING PER OUR PRACTICES

BEAM PROPERTIES

A = 879 ~~sq~~in.^{2}
Wt = 915 lb/ft
Y_{BB} = 18.17 in.
Y_{TB} = 17.83 in.
Ix = 145772 in.^{4}
Iy = 102920 in.^{4}
S_{BB} = 8023.3 in.^{3}
S_{TB} = 8175.1 in.^{3}

~~Vol/Sur = 4.146 in~~

ORDER: A I_x I_y S_{TB} S_{BB} Y_{BB} Y_{TB} Wt.

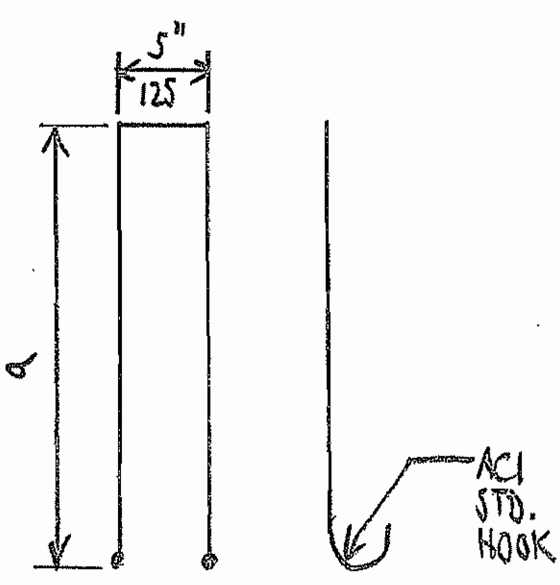
HYBRID BULB-TEE BEAM,
TYPE BT 36 x 49 / 914 x 1245

Figure 63-14Y(1)

HYBRID BULB-TEES
VARIABLE-SIZED BENT MILD REINFORCEMENT

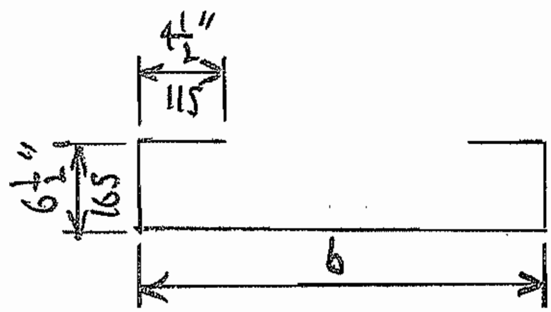
ENGLISH UNITS				
BEAM DESIG.	402		403	
	a	Total Lgth.	*b*	Total Lgth.
36 x 49	3'-6"	8'-5"	3'-11"	5'-9"
42 x 49	4'-0"	9'-5"	3'-11"	5'-9"
60 x 61	5'-6"	12'-5"	4'-11"	6'-9"
66 x 61	6'-0"	13'-5"	4'-11"	6'-9"
METRIC UNITS				
BEAM DESIG.	1302		1303	
	a	Total Lgth.	*b*	Total Lgth.
914 x 1245	1064	2553	1195	1755
1067 x 1245	1217	2859	1195	1755
1524 x 1550	1674	3773	1500	2060
1829 x 1550	1979	4383	1500	2060

Beam designation is height times top-flange width.
English units, inches; metric units, millimeters

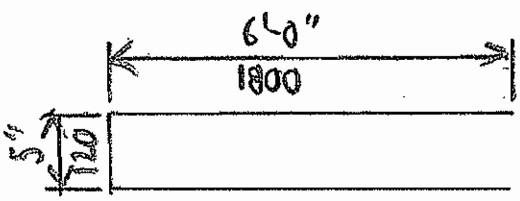


END VIEW   SIDE VIEW

402 X ____
1302 X ____

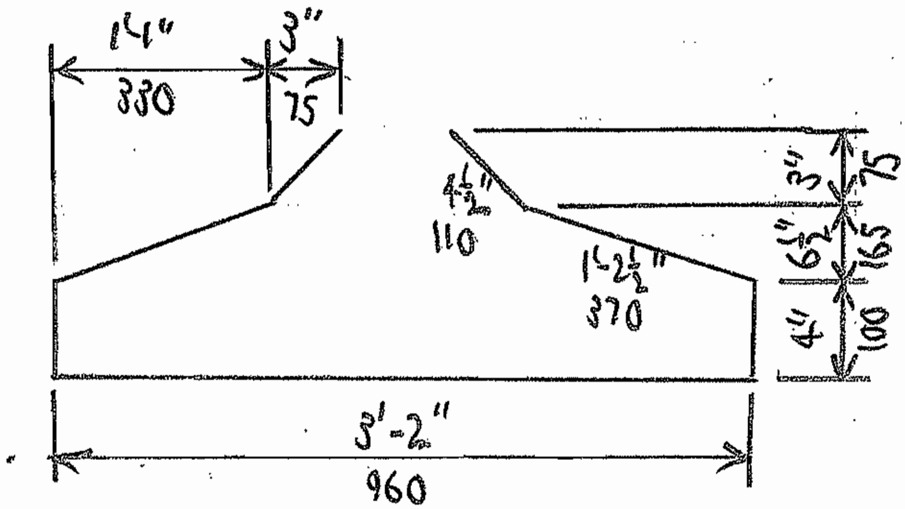


403 X ____
1303 X ____

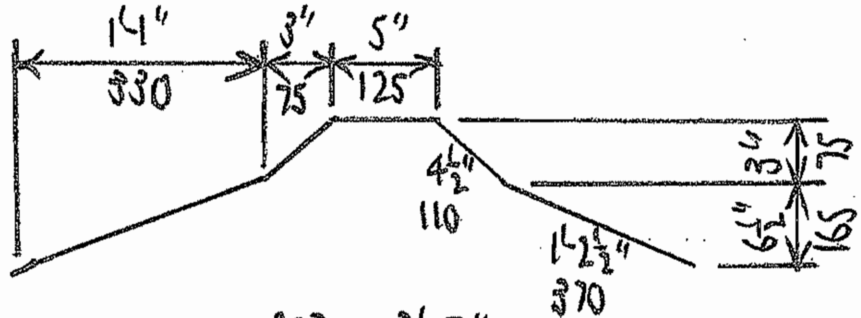


401 X 12'-5"
1301 X 3720

FIG. 63-14Y(5) CONT.



301 x 7'-0"
1001 x 2120



302 x 3'-7"
1002 x 1080

FIG. 63-14Y(S) CONT.

MTG 39, AGEN ITEM 11
ATTACHMENT No. 4

61-2.0 STRIP METHOD

61-2.02 Application of the Strip Method to a Composite Concrete Deck

61-2.02(01) Patch Loading

2. Negative Moment.

Summarizing, the maximum negative moment and accompanying reaction at the center of support are computed using concentrated wheel loads. *LRFD* Article 4.6.2.1.6 specifies the location of the negative moment design section as follows:

a. at the face of support for concrete box beams;
b. one-quarter of the flange width from centerline of support for steel beams; or
c. one-third of the flange width, not to exceed 15 in. (380 mm) from the centerline of support, for precast I-shaped and T-shaped concrete beams.

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APPROVED DES-MAN FIG? — NO.

2007, 4TH ED.

4.6.2.1.5 Distribution of Wheel Loads

If the spacing of supporting components in the secondary direction exceeds 1.5 times the spacing in the primary direction, all of the wheel loads shall be considered to be applied to the primary strip, and the provisions of Article 9.7.3.2 may be applied to the secondary direction.

If the spacing of supporting components in the secondary direction is less than 1.5 times the spacing in the primary direction, the deck shall be modeled as a system of intersecting strips.

The width of the equivalent strips in both directions may be taken as specified in Table 4.6.2.1.3-1. Each wheel load shall be distributed between two intersecting strips. The distribution shall be determined as the ratio between the stiffness of the strip and the sum of stiffnesses of the intersecting strips. In the absence of more precise calculations, the strip stiffness, k_s, may be estimated as:

$$k_s = \frac{EI_s}{S^3} \qquad (4.6.2.1.5\text{-}1)$$

where:

I_s = moment of inertia of the equivalent strip (in.^{4})

S = spacing of supporting components (in.)

C4.6.2.1.5

This Article attempts to clarify the application of the traditional AASHTO approach with respect to continuous decks.

4.6.2.1.6 Calculation of Force Effects

The strips shall be treated as continuous beams or simply supported beams, as appropriate. Span length shall be taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components shall be assumed to be infinitely rigid.

The wheel loads may be modeled as concentrated loads or as patch loads whose length along the span shall be the length of the tire contact area, as specified in Article 3.6.1.2.5, plus the depth of the deck. The strips should be analyzed by classical beam theory.

The design section for negative moments and shear forces, where investigated, may be taken as follows:

- For monolithic construction, closed steel boxes, closed concrete boxes, open concrete boxes without top flanges, and stemmed precast beams, i.e., Cross-sections (b), (c), (d), (e), (f), (g), (h), (i), and (j) from Table 4.6.2.2.1-1, at the face of the supporting component,

- For steel I-beams and steel tub girders, i.e., Cross-sections (a) and (c) from Table 4.6.2.2.1-1, one-quarter the flange width from the centerline of support,

C4.6.2.1.6

This is a deviation from the traditional approach based on a continuity correction applied to results obtained for analysis of simply supported spans. In lieu of more precise calculations, the unfactored design live load moments for many practical concrete deck slabs can be found in Table A4-1.

For short-spans, the force effects calculated using the footprint could be significantly lower, and more realistic, than force effects calculated using concentrated loads.

Reduction in negative moment and shear replaces the effect of reduced span length in the current code. The design sections indicated may be applied to deck overhangs and to portions of decks between stringers or similar lines of support.

Past practice has been to not check shear in typical decks. A design section for shear is provided for use in nontraditional situations. It is not the intent to investigate shear in every deck.

- For precast I-shaped concrete beams and open concrete boxes with top flanges, i.e., Cross-sections (c) and (k) from Table 4.6.2.2.1-1, one-third the flange width, but not exceeding 15.0 in., from the centerline of support,

- For wood beams, i.e., Cross-section (l) from Table 4.6.2.2.1-1, one-fourth the top beam width from centerline of beam.

For open box beams, each web shall be considered as a separate supporting component for the deck. The distance from the centerline of each web and the adjacent design sections for negative moment shall be determined based on the type of construction of the box and the shape of the top of the web using the requirements outlined above.

4.6.2.1.7 Cross-Sectional Frame Action

Where decks are an integral part of box or cellular cross-sections, flexural and/or torsional stiffnesses of supporting components of the cross-section, i.e., the webs and bottom flange, are likely to cause significant force effects in the deck. Those components shall be included in the analysis of the deck.

If the length of a frame segment is modeled as the width of an equivalent strip, provisions of Articles 4.6.2.1.3, 4.6.2.1.5, and 4.6.2.1.6 may be used.

C4.6.2.1.7

The model used is essentially a transverse segmental strip, in which flexural continuity provided by the webs and bottom flange is included. Such modeling is restricted to closed cross-sections only. In open-framed structures, a degree of transverse frame action also exists, but it can be determined only by complex, refined analysis.

In normal beam-slab superstructures, cross-sectional frame action may safely be neglected. If the slab is supported by box beams or is integrated into a cellular cross-section, the effects of frame action could be considerable. Such action usually decreases positive moments, but may increase negative moments resulting in cracking of the deck. For larger structures, a three-dimensional analysis may be appropriate. For smaller structures, the analysis could be restricted to a segment of the bridge whose length is the width of an equivalent strip.

Extreme force effects may be calculated by combining the:

- Longitudinal response of the superstructure approximated by classical beam theory, and

- Transverse flexural response modeled as a cross-sectional frame.

ATTACHMENT No. 5

The sloping portion of the curves represents the braking force that includes a portion of the lane load. This represents the possibility of having multiple lanes of vehicles contributing to the same braking event on a long bridge. Although the probability of such an event is likely to be small, the inclusion of a portion of the lane load gives such an event consideration for bridges with heavy truck traffic and is consistent with other design codes.

Because the LRFD braking force is significantly higher than that required in the Standard Specifications, this issue becomes important in rehabilitation projects designed under previous versions of the design code. In cases where substructures are found to be inadequate to resist the increased longitudinal forces, consideration should be given to design and detailing strategies which distribute the braking force to additional substructure units during a braking event.

3.6.5 Vehicular Collision Force: *CT*

3.6.5.1 Protection of Structures

The provisions of Article 3.6.5.2 need not be considered for structures which are protected by:

- An embankment;
- A structurally independent, crashworthy ground-mounted 54.0-in. high barrier, located within 10.0 ft. from the component being protected; or
- A 42.0-in. high barrier located at more than 10.0 ft. from the component being protected.

In order to qualify for this exemption, such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.

C3.6.5.1

For the purpose of this Article, a barrier may be considered structurally independent if it does not transmit loads to the bridge.

Full-scale crash tests have shown that some vehicles have a greater tendency to lean over or partially cross over a 42.0-in. high barrier than a 54.0-in. high barrier. This behavior would allow a significant collision of the vehicle with the component being protected if the component is located within a few ft. of the barrier. If the component is more than about 10.0 ft. behind the barrier, the difference between the two barrier heights is no longer important.

3.6.5.2 Vehicle and Railway Collision with Structures

Unless protected as specified in Article 3.6.5.1, abutments and piers located within a distance of 30.0 ft. to the edge of roadway, or within a distance of 50.0 ft. to the centerline of arailway track, shall be designed for an equivalent static force of 400 kip, which is assumed to act in any direction in a horizontal plane, at a distance of 4.0 ft. above ground.

The provisions of Article 2.3.2.2.1 shall apply.

C3.6.5.2

It is not the intent of this provision to encourage unprotected piers and abutments within the setbacks indicated, but rather to supply some guidance for structural design when it is deemed totally impractical to meet the requirements of Article 3.6.5.1.

The equivalent static force of 400 kip is based on the information from full-scale crash tests of barriers for redirecting 80.0-kip tractor trailers and from analysis of other truck collisions. The 400-kip train collision load is based on recent, physically unverified, analytical work (*Hirsch 1989*). For individual column shafts, the 400-kip load should be considered a point load. For wall piers, the load may be considered to be a point load or may be distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft. wide by 2.0 ft. high. These dimensions were determined by considering the size of a truck frame.

3.6.5.3 Vehicle Collision with Barriers

The provisions of Section 13 shall apply.

3.7 WATER LOADS: *WA*

3.7.1 Static Pressure

Static pressure of water shall be assumed to act perpendicular to the surface that is retaining the water. Pressure shall be calculated as the product of height of water above the point of consideration and the specific weight of water.

Design water levels for various limit states shall be as specified and/or approved by the Owner.

3.7.2 Buoyancy

Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures, as specified in Article 3.7.1, acting on all components below design water level.

C3.7.2

For substructures with cavities in which the presence or absence of water cannot be ascertained, the condition producing the least favorable force effect should be chosen.

3.7.3 Stream Pressure

3.7.3.1 Longitudinal

The pressure of flowing water acting in the longitudinal direction of substructures shall be taken as:

$$p = \frac{C_D V^2}{1{,}000} \tag{3.7.3.1-1}$$

where:

p = pressure of flowing water (ksf)

C_D = drag coefficient for piers as specified in Table 1

V = design velocity of water for the design flood in strength and service limit states and for the check flood in the extreme event limit state (ft./sec.)

Table 3.7.3.1-1 Drag Coefficient.

Type	C_D
semicircular-nosed pier	0.7
square-ended pier	1.4
debris lodged against the pier	1.4
wedged-nosed pier with nose angle 90° or less	0.8

C3.7.3.1

For the purpose of this Article, the longitudinal direction refers to the major axis of a substructure unit.

The theoretically correct expression for Eq. 1 is:

$$p = C_D \frac{w}{2g} V^2 \tag{C3.7.3.1-1}$$

where:

w = specific weight of water (kcf)

V = velocity of water (ft./sec.)

g = gravitational acceleration constant—32.2 (ft./sec.^{2})

As a convenience, Eq. 1 recognizes that $w/2g \sim 1/1{,}000$, but the dimensional consistency is lost in the simplification.

A13.1.2 Anchorages

The yield strength of anchor bolts for steel railing shall be fully developed by bond, hooks, attachment to embedded plates, or any combination thereof.

Reinforcing steel for concrete barriers shall have embedment length sufficient to develop the yield strength.

CA13.1.2

Noncorrosive bonding agents for anchor dowels may be cement grout, epoxy, or a magnesium phosphate compound. Sulfur or expansive-type grouts should not be used.

Some bonding agents on the market have corrosive characteristics; these should be avoided.

Development length for reinforcing bars is specified in Section 5.

A13.2 TRAFFIC RAILING DESIGN FORCES

Unless modified herein, the extreme event limit state and the corresponding load combinations in Table 3.4.1-1 shall apply.

Railing design forces and geometric criteria to be used in developing test specimens for a crash test program should be taken as specified in Table 1 and illustrated in Figure 1. The transverse and longitudinal loads given in Table 1 need not be applied in conjunction with vertical loads.

The effective height of the vehicle rollover force is taken as:

$$H_e = G - \frac{12WB}{2F_t} \qquad \text{(A13.2-1)}$$

where:

G = height of vehicle center of gravity above bridge deck, as specified in Table 13.7.2-1 (in.)

W = weight of vehicle corresponding to the required test level, as specified in Table 13.7.2-1 (kips)

B = out-to-out wheel spacing on an axle, as specified in Table 13.7.2-1 (ft.)

F_t = transverse force corresponding to the required test level, as specified in Table 1 (kips)

Railings shall be proportioned such that:

$$\overline{R} \geq F_t \qquad \text{(A13.2-2)}$$

$$\overline{Y} \geq \frac{H_e}{12} \qquad \text{(A13.2-3)}$$

in which:

$$\overline{R} = \Sigma R_i \qquad \text{(A13.2-4)}$$

$$\overline{Y} = \frac{\Sigma(R_i Y_i)}{\overline{R}} \qquad \text{(A13.2-5)}$$

CA13.2

Nomenclature for Eqs. 1 and 2 is illustrated in Figure C1.

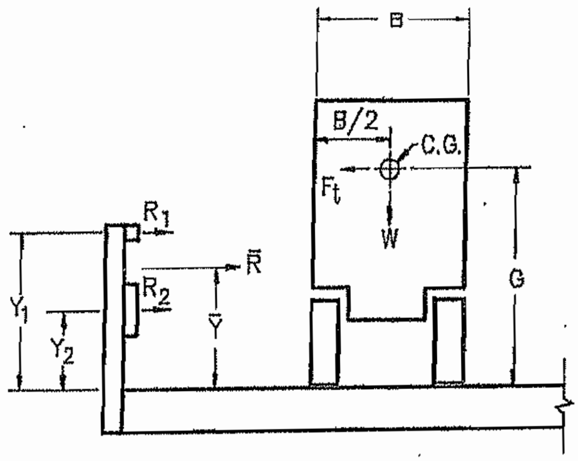

Figure CA13.2-1 Traffic Railing.

If the total resistance, $\overline{R}$, of a post-and-beam railing system with multiple rail elements is significantly greater than the applied load, F_t, then the resistance, R_i, for the lower rail element(s) used in calculations may be reduced.

The reduced value of $\overline{R}$ will result in an increase in the computed value of $\overline{Y}$. The reduced notional total rail resistance and its effective height must satisfy Eqs. 2 and 3.

where:

R_i = resistance of the rail (kips)

Y_i = distance from bridge deck to the ith rail (ft.)

All forces shall be applied to the longitudinal rail elements. The distribution of longitudinal loads to posts shall be consistent with the continuity of rail elements. Distribution of transverse loads shall be consistent with the assumed failure mechanism of the railing system.

Eq. 1 has been found to give reasonable predictions of effective railing height requirements to prevent rollover.

If the design load located at H_e falls between rail elements, it should be distributed proportionally to rail elements above and below such that $Y \geq H_e$.

As an example of the significance of the data in Table 1, the length of 4.0 ft. for L_t and L_L is the length of significant contact between the vehicle and railing that has been observed in films of crash tests. The length of 3.5 ft. for TL-4 is the rear-axle tire diameter of the truck. The length of 8.0 ft. for TL-5 and TL-6 is the length of the tractor rear tandem axles: two 3.5-ft. diameter tires, plus 1.0 ft. between them.

F_v, the weight of the vehicle lying on top of the bridge rail, is distributed over the length of the vehicle in contact with the rail, L_v.

For concrete railings, Eq. 1 results in a theoretically-required height, H, of 34.0 in. for Test Level TL-4. However, a height of 32.0 in., shown in Table 1, was considered to be acceptable because many railings of that height have been built and appear to be performing acceptably.

The minimum height, H, listed for TL-1, TL-2, and TL-3 is based on the minimum railings height used in the past. The minimum effective height, H_e, for TL-1 is an estimate based on the limited information available for this test level.

The minimum height, H, of 42.0 in., shown in Table 1, for TL-5 is based on the height used for successfully crash-tested concrete barrier engaging only the tires of the truck. For post and beam metal bridge railings, it may be prudent to increase the height by 12.0 in. so as to engage the bed of the truck.

The minimum height, H, shown in Table 1, for TL-6 is the height required to engage the side of the tank as determined by crash test.

Table A13.2-1 Design Forces for Traffic Railings.

Design Forces and Designations	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips) Down	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft.)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft.)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum H Height of Rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0