Structural Engineering Guidance No. 24-02

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SUBJECT: Seismic Design Procedure FOR Mass Inertial Forces AT INTEGRAL ABUTMENTS

Contact: Darren Kemna, Suresh Patel

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1. Background and Purpose:

Current policy for LFD Design is to ignore seismic design of integral abutments for single span bridges and multi-span bridges where only seismic details are required. The new policy outlined in [SEG 24-01](https://modotgov.sharepoint.com/%3Aw%3A/r/sites/CO_BR/Shared%20Documents/General/Development/Structural%20Engineering%20Guidance%20SEG/Active%20SEG/24-01-SEG%20New%20LRFD%20Seismic%20Design%20Philosophy.docx?d=w561de6809d4544c8b0cab63220cbfdc3&csf=1&web=1&e=txMxls) for LRFD design does not require a complete seismic analysis for single span bridges, or multi-span bridges with SDC B, but on important routes the abutments and potentially the foundation shall be designed for the mass inertial forces per the procedure below. The concept is adapted from the SGS requirement for single span bridges and expanded to multi-span bridges in SDC B on important routes. Designing the abutment to behave elastically for mass inertial forces will help to limit the demand on the intermediate bents which are still required to receive seismic details.

1. Applicable Bridges

Currently, the following design procedure is required for single span bridges on major routes or 1st or 2nd priority earthquake routes with a final SDC B, C or D. Similarly, the design procedure is required for multi-span bidges on major routes or 1st or 2nd priority earthquake routes with a final SDC B. This procedure is required for integral abutments on LRFD design projects where wings, CIP retaining walls or MSE walls are used to retain the backfill. Regardless of whether this procedure is used to design the abutment, seismic analysis provisions shall not be ignored for walls (detached wing walls, CIP retaining walls and MSE walls) that support abutment fill, but the procedure to design walls is not discussed herein. See the Bridge Seismic Design Flowchart for the latest applicability requirements.

Note: The procedure outlined herein is for the design of integral abutments. Similar design forces shall be used for non-integral abutments assuming the joint to be closed.

1. Integral Abutment Seismic Design Procedure for Mass Inertial Forces

The lateral seismic loads due to the inertial force from the superstructure mass shall be transmitted to the abutment per AASHTO Guide Specifications for LRFD Seismic Bridge Design (SGS), 3rd edition, article 4.5 and as outlined below.

* **Compute the lateral inertial forces, PEQT andPEQL**

As = design response spectral acceleration coefficient at a period of zero seconds

 ≤ 0.75 … practical design limit

**Transverse Direction:**

Assume each abutment resists the transverse seismic force proportional to the dead load reaction from the static analysis.

**PEQT** = AsPDLT

Where,

PEQT = Transverse inertial force at abutment

PDLT = superstructure dead load reaction at abutment plus self weight of abutment. Approach slab weight and pile self weight shall be ignored.

**Longitudinal Direction:**

Assume the abutment resists the longitudinal seismic force proportional to dead load reaction at both the abutment and the adjacent bent.

**PEQL** = AsPDLL

Where,

PEQL = Longitudinal inertial force

PDLL = superstructure dead load reaction at abutment and adjacent bent plus self weight of abutment(s). Approach slab weight and pile self weight shall be ignored.

* **Concrete Diaphragm Design**

***Passive Resistance of Backfill Normal to Wall (PPN):***

The soil behind the abutment wall is considered in the longitudinal movement of the bridge during a seismic event. For skewed abutments the soil mass is assumed to resist movement normal to the wall.

**PPN** = PPHwWw

Where,

PPN = Passive soil resistance on abutment wall, kips

PP = (2/3)HW = Passive lateral earth pressure behind backwall (ksf) per foot width of backwall

HW = Height of abutment wall against soil backfill, ft (do not include approach slab thickness)

WW = Width of abutment wall along skew, ft

***Design Loads and Resistances:***

Conservatively assume 100% of PEQL force in longitudinal direction will be resisted by abutment wall. For skewed abutments 100% of PEQL shall be applied normal to the wall. Determine whether the full passive soil resistance, PPN, is activated by comparing to the inertial force, PEQL. If the inertial force is greater than the passive soil resistance then the passive soil resistance shall be used. Otherwise the design force shall not be taken to be greater than the inertial force.

Soil Pressure,

sN = Min{PEQL,PPN}/(HWWW)

For horizontal reinforcement design ignore restraint from piles and assume fixed-end span between girders. For completeness the beam cap may be checked by assuming a similarly fixed-end span between piles, but typically will not control the flexural design and if it does control the flexural design it is unlikely to require additional reinforcement compared to the standard. The required size and spacing of horizontal bars shall be used in both faces of diaphragm and beam cap. For shear design the resistance from reinforcing steel shall be ignored.

Design Moment,

Mu = sNs2/12… per foot height of abutment wall

Factored Moment Resistance,

fMn = fAsFye(d-a/2)

Where,

 s = girder spacing along skew

 f = 1.0 = resistance factor for extreme event

 As = Area of reinforcing steel per foot height, (Use #6 @ 12” min.)

 Fye = 68 ksi = expected yield strength of reinforcing steel

 d = effective depth of tensile reinforcement (front face)

 a = AsFye/(0.85f’ceb)

 f’ce = 1.3f’c = expected concrete strength of diaphragm

Design Shear,

Vu = sNs/2

Factored Shear Resistance,

fVn = f0.0316b(f’ce)0.5bvdv

Where,

 s = girder spacing along skew

 f = 1.0 = resistance factor for extreme event

 f’ce = 1.3f’c = expected concrete strength of diaphragm

 bv = 12 in.

 dv = Min{Mn/(AsFye), 0.9d, 0.72h}

 $β=\frac{4.8}{(1+750ε\_{s})}\frac{51}{(39+ d\_{v})}$ , $ε\_{s}= \frac{\frac{M\_{u}}{d\_{v}}+V\_{u}}{E\_{s}A\_{s}}$

 Es = 29,000 ksi

 h = width of diaphragm normal to horizontal reinforcement

For vertical reinforcement design assume pinned restraint at bottom of beam and bottom of approach slab. Restraint at top comes from the bridge slab (not approach slab), but pinned assumption is conservative and simplifies the design procedure. Resistance calculations are similar to horizontal reinforcement. Use #5 @ 12” minimum stirrups between girders.

Design Moment, Mu = sNHw2/8… per foot width of abutment wall

Design Shear, Vu = sNHw/2

sN

WW

s

CL girder

Fig. 1 – Abutment wall loading – Plan view

sN

HW

Fig. 2 – Abutment wall loading – Section thru

* **Wing Design**

***Passive resistance of Backfill in transverse direction (Ppt):***

The soil on either side of the wing is considered in the transverse movement of the bridge during a seismic event. The passive resistance of the soil backfill is considered two-third effective against resisting movement from the wing wall. The passive resistance of the exterior slope is considered one-half as effective as the interior soil backfill.

*Interior face of wing:*

Ppti = (2/3)(2/3)HiAi

Where,

Ppti = Passive soil resistance on interior face of wing, kips

Hi = Height of wing wall against soil backfill (do not include approach slab thickness), ft

(2/3)Hi = Passive lateral earth pressure, ksf

Ai = Area of wing against soil backfill (exclude corner brace and approach slab), sq. ft

*Exterior face of wing:*

Ppte = (1/3)(2/3)HiAi

Where,

Ppte = Passive soil resistance on exterior face of wing, kips

Hi,Ai, and (2/3)Hi = same as above

Transverse passive resistance, Ppt = Ppti +Ppte = (2/3)HiAi

***Design Loads and Resistances:***

Assume the inertial force in transverse direction will be distributed equally to each wing at the abutment. The total inertial force, PEQT, should be compared to the transverse passive resistance, Ppt. If PEQT is less than Ppt then use the inertial force distributed equally to each wing to calculate the design soil pressure. If PEQT is greater than Ppt then the passive resistance, Ppti, is used to calculate the design soil pressure. Both forces are assumed to be uniformly distributed to the fill face of the wing past the corner brace.

Design force,

If PEQT ≤ PPt then PT = (1/2)PEQT otherwise PT = Ppti…(2 wings)

Design Soil Pressure,

sT = PT/Ai

For horizontal reinforcement design assume a fixed restraint at the edge of the corner brace at wing. The required size and spacing of bars in the interior face of wing should also be used in the exterior face. The portion of the wing below the construction joint will control due to the lower specified concrete strength. Development of horizontal reinforcement shall be considered at the edge of corner brace.

Design Moment,

Mu = sTL2/2… per foot height of wing wall

Factored Moment Resistance,

fMn = fAsFye(d-a/2)

Where,

 L = wing length past corner brace

 f = 1.0 = resistance factor for extreme event

 As = Area of reinforcing steel per foot height, (Use #6 @ 8” min.)

 Fye = 68 ksi = expected yield strength of reinforcing steel

 d = effective depth of horizontal reinforcement

 a = AsFye/(0.85f’ceb)

 f’ce = 1.3f’c = expected concrete strength of wing below construction joint

Design Shear,

Vu = sTL

Factored Shear Resistance,

fVn = f0.0316b(f’ce)0.5bvdv

Where,

 L = wing length past corner brace

 f = 1.0 = resistance factor for extreme event

 f’ce = 1.3f’c = expected concrete strength of wing below construction joint

 bv = 12 in.

 dv = Min{Mn/(AsFye), 0.9d, 0.72h}

 $β=\frac{4.8}{(1+750ε\_{s})}\frac{51}{(39+ d\_{v})}$ , $ε\_{s}= \frac{\frac{M\_{u}}{d\_{v}}+V\_{u}}{E\_{s}A\_{s}}$ , Es = 29,000 ksi

 h = wing thickness

For vertical reinforcement use the standard #6 @ 12”.

If wing cannot meet shear design or flexural design then increase corner brace, wing thickness or add interior wing to reduce load. If an interior wing is added the design force evaluation is modified to…

If PEQT ≤ PPt then PT = (1/3)PEQT otherwise PT = PPti …(3 wings)

sT

sT

Fig. 3 – Wing loading – Plan view

Ignore approach slab thickness

Ai

Ignore corner brace

Fig. 4 – Wing loading area on interior face

* **Pile Design**

Pile design need only be considered when the abutment fill is supported primarily by a MSE wall or CIP retaining wall (i.e., no wingwalls). Piles shall be checked for shear and combined axial-flexure.

***Pile Inertial Forces (PH)***

Similar to the diaphragm design the longitudinal inertial force should be applied normal to the bent. Unlike the wing design, the transverse inertial force should be applied parallel to the bent. For pile design consider 70% of PEQL is resisted by the abutment wall normal to the bent. If a MSE wall or CIP retaining wall is used instead of wings then assume 100% of PEQT is transferred to the piles parallel to bent.

Long. inertial force per pile applied normal to bent,

PHL = (30% PEQL)/(no. of piles at abutment)

Trans. inertial force per pile applied parallel to bent,

PHT = PEQT/(no. of piles at abutment)

The following load combinations should be considered for pile design.

Case 1 = 1.0PHL + 0.3PHt

Case 2 = 0.3PHL + 1.0PHt

*CIP piles:*

Resultant horizontal force at top of pile,

PH = Max{(PHT2 + (0.3PHL)2)1/2 , ((0.3PHT)2 + PHL2)1/2}

*Steel HP Piles:*

Horizontal force at top of pile assuming standard pile orientation,

PH = PHT … strong-axis bending

PH = PHL … weak-axis bending

***Pile Shear Resistance***

The pile inertial shear force, PH, shall be checked against the nominal shear resistance, Vn. The resistance factor for shear at the extreme event limit state shall be taken as 1.

*CIP piles:*

Nominal shear resistance of CFST,

Vn = 1.15DtFye + 0.0316b(f’ce)0.5Ac

Where,

 D = outer diameter of steel tube

 t = wall thickness of steel tube

 Fye = 1.5\*45 ksi = 67.5 ksi = expected yield strength of steel tube

 b = 6.0

 f’ce = 1.3f’c = 1.3\*4.0 ksi = 5.2 ksi = expected concrete strength of concrete core

 Ac = cross-sectional area of concrete core

*Steel HP Piles:*

Shear resistance need only be checked for forces applied parallel to web of pile.

Nominal shear resistance for web of H-pile,

Vn = Vp = 0.58FyeDtw

Where,

 D = web depth

 tw = web thickness

 Fye = 1.1\*50 ksi = 55 ksi = expected yield strength of A709 Grade 50 steel

***Pile Axial-Flexural Resistance***

The axial compressive load, Pu shall be taken as the axial load from the static dead load analysis, PDL.

Use Lpile model to determine the maximum bending moment (typically at top of pile). For an elastic analysis apply PDL and PH with slope = 0 rad and determine Mu from LPile output. For H-piles determine the maximum bending moment for strong-axis and weak-axis bending. Liquefaction effect shall be considered by modelling a liquefiable layer or removing the soil layer in the LPile model. This requirement is waived if a liquefaction analysis is not provided in the Foundation Investigation Geotechnical Report per current MoDOT seismic guidelines (see [Bridge Seismic Planning Flowchart](https://modotgov.sharepoint.com/%3Ab%3A/r/sites/DE/EPG/BR/Bridge_Seismic_Planning_Flowchart.pdf?csf=1&web=1&e=X0ObZD)).

Verify that interaction ratio for axial and bending is less than 1.0 per [LRFD 6.9.2.2](https://modotgov.sharepoint.com/%3Ab%3A/r/sites/cm/AASHTOepub/LRFDBDS-9.pdf?csf=1&web=1&e=RXcYsb).

*CIP piles:*

The factored axial compressive resistance, Pr may be taken equal to the PNDC as outlined in EPG 751.36.5.5. The flexural resistance, Mr shall use either the provisions for concrete-filled tubes or CFSTs in accordance with LRFD 6.12.2.3.2 or 6.12.2.3.3 respectively. The resistance factor for compression and flexure at the extreme event limit state shall be taken as 1.

*Steel HP Piles:*

The factored axial compressive resistance, Pr may be taken equal to the PNDC as outlined in EPG 751.36.5.5. For strong-axis bending the elastic flexural resistance shall be used for Mr. For weak-axis bending the plastic flexural resistance may be used. The resistance factor for compression and flexure at the extreme event limit state shall be taken as 1.

If pile design cannot meet the requirements then increase pile size or add piles.

PDL

PH

Fig. 5 – Typical Pile model at integral abutments