

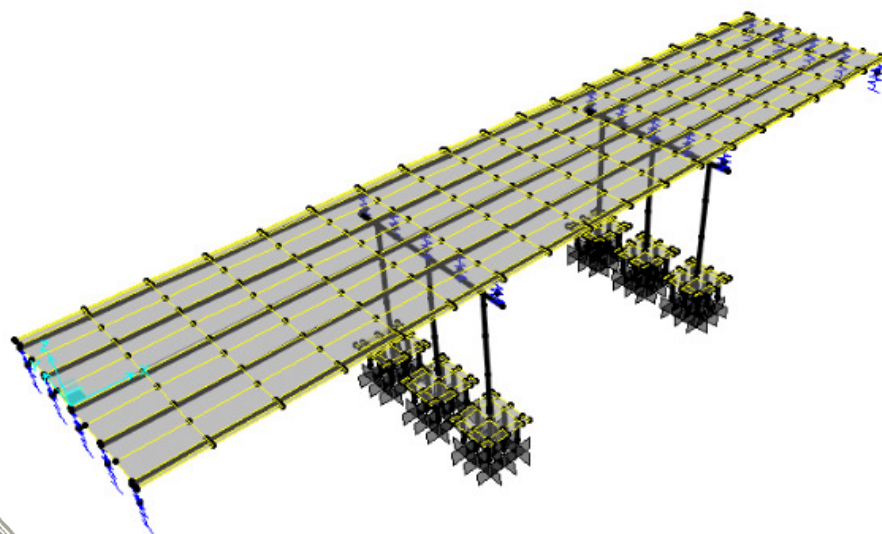


Bridge Design Manual

Chapter 5

Seismic Design and Retrofit

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Chapter 5 Seismic Design and Retrofit

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Chapter 5**Seismic Design and Retrofit**

5.1 Design Criteria

Seismic design of new bridges and bridge widenings shall conform to *AASHTO 3.10 and AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Analysis and design of seismic retrofits for existing bridges shall be completed in accordance with Section 5.3 of this chapter. For nonconventional bridges, bridges that are deemed critical or essential, or bridges that fall outside the scope of the *AASHTO LRFD Specifications* for any other reason, project-specific design requirements shall be developed and submitted to the Bridge Design Chief for approval.

5.1.1 Design Approach and Considerations

The Department's primary objective when designing bridges to resist earthquake loading is to minimize damage and prevent the loss of span or collapse of a structure. The AASHTO seismic design requirements are not intended to eliminate all damage.

The primary objective in seismic design is to prevent foundation failure, liquefaction failure, support failure at the bearings and expansion joints within a span, shear or moment failure of the columns, and failure of the connecting components. Based on the bridge classification and the performance objectives, it is the Department's policy to permit designs with inelastic deformation that is ductile. Damage is permitted, provided it is at locations that can be readily inspected and repaired after the seismic event. Plastic hinging in superstructure components (which may impact the gravity load support capabilities of the structure) or bearing systems not capable of providing the expected seismic displacement and forces is not permitted.

All bridge components and their foundation systems must provide the means of adequately dissipating energy or must be capable of sufficiently resisting seismically induced structure component displacements. The systems must provide for uninterrupted load paths for transmitting seismically induced forces into the ground.

Some basic seismic design concepts that will improve seismic performance by enhancing the load distribution to the substructures are: continuity of the superstructure, symmetry in structure stiffness and geometry, and overall structure redundancy. These design concepts not only benefit the seismic performance but also the overall performance of the structure.

See [Appendix 5.1-A1](#) for AASHTO's seismic design flowchart.

5.1.2 Bridge Classification and Performance Objectives**A. "Regular" and "Irregular" Bridges**

The definitions of "regular" and "irregular" bridges for seismic design are given in *AASHTO Table 4.7.4.3.1-2*. Generally, bridges are regular if there are 2 to 6 spans (single span bridges are always regular), do not have span length ratios that are significantly large, do not have pier stiffness ratios that are significantly large, and are not overly curved. Multi-span bridges defined as "regular" or "irregular" shall be designed for analysis requirements noted in *AASHTO 4.7.4.3*.

B. Operational Classification

The definitions of operational categories, “Critical, Essential or Other” for seismic design are given in *AASHTO 3.10.5*. The Department will classify all bridges on a case-by-case basis.

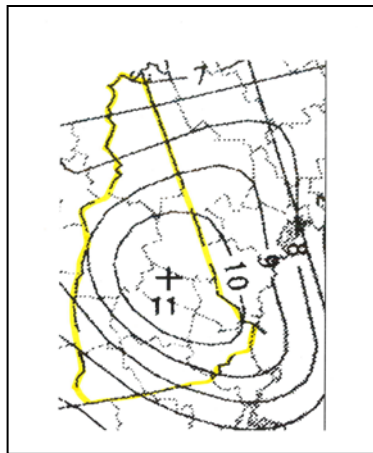
Most bridges on the State System and many on the Municipal System should be classified as “Essential”. Some Municipal bridges may be classified as “Other”. Major bridges fitting the classifications of either “Critical” or “Essential” will be so designated by the Chief of Bridge Design.

5.2 Seismic Analysis and Design Procedure

The bridge classifications described in Section 5.1.2, in conjunction with the site classifications as described in Section 5.2.4, form the basis for the selection of the seismic analysis, design, and detailing procedures. These procedures outline the specific requirements for analysis and design reflecting the variation in seismic risk.

5.2.1 Peak Ground Acceleration Coefficient (PGA)

In accordance with AASHTO Figure 3.10.2.1-1, the peak ground acceleration coefficient for New Hampshire varies on the map from 6.3% - 11%.

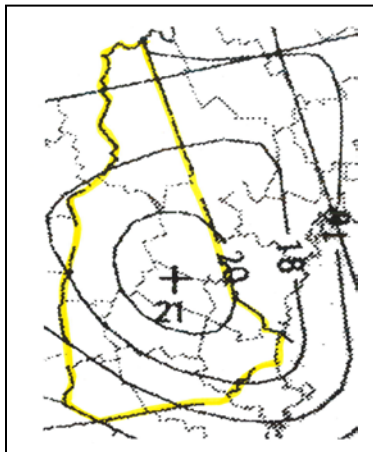


AASHTO Peak Ground Acceleration Coefficient (PGA) Map

Figure 5.2.1-1

5.2.2 Short-Term Spectral Acceleration Coefficient (S_S)

In accordance with AASHTO Figure 3.10.2.1-2, the short-term spectral acceleration coefficient for New Hampshire varies on the map from 14% - 21%.

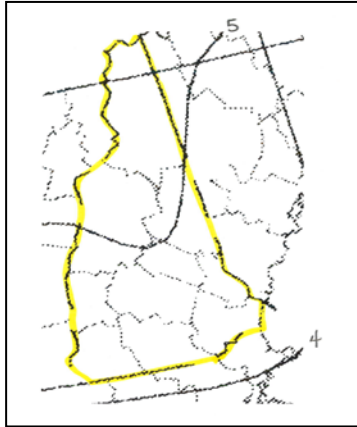


AASHTO Short-Term Spectral Acceleration Coefficient (S_S) Map

Figure 5.2.2-1

5.2.3 Long-Term Spectral Acceleration Coefficient (S_1)

In accordance with *AASHTO Figure 3.10.2.1-3*, the long-term spectral acceleration coefficient for New Hampshire varies on the map from approximately **4.15% - 5.5%**.



AASHTO Long-Term Spectral Acceleration Coefficient (S_1) Map

Figure 5.2.3-1

5.2.4 Site Classifications

The Bureau of Materials and Research will provide the site classification in the Geotechnical Report as defined in *AASHTO Table 3.10.3.1-1*. The majority of New Hampshire will be in site class A-D.

- Soils characteristics corresponding to Site Class E & F will require a site-specific evaluation.
- If the soil characteristics vary, the worst soil classification will be used for seismic classification.
- The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or geologist/seismologist based on similar competent rock with moderate fracturing and weathering. Unless shear wave velocity is measured on site, soft, highly-fractured, and weathered rock shall be classified as Site Class C.
- The hard rock category, Site Class A, shall be supported by shear wave velocity measurements on site or shall be estimated by a geotechnical engineer or geologist/seismologist base on profiles of the same rock type in the same formation with an equal degree of weathering and fracturing.

5.2.5 Site Factors

A. Site Factor at Zero-Period (F_{pga})

- Section 5.2.1 shows the maximum peak ground acceleration coefficient for New Hampshire: $PGA_{max} = 0.11$
- The F_{pga} can be determined from column “PGA < 0.10” in accordance with *AASHTO Table 3.10.3.2-1*.

B. Site Factor at Short-Period Range (F_a)

- Section 5.2.2 shows that the maximum short term spectral acceleration coefficient for New Hampshire: $S_{s \max} = 0.21$
- The F_a can be determined from column “ $S_s < 0.25$ ” in accordance with *AASHTO Table 3.10.3.2-2*.

C. Site Factor at Long-Period Range (F_v)

- Section 5.2.3 shows that the maximum long term spectral acceleration coefficient for New Hampshire: $S_{1 \max} = 0.055$
- The F_v can be determined from column “ $S_1 < 0.1$ ” in accordance with *AASHTO Table 3.10.3.2-3*.

5.2.6 Design Response Spectrum

For locations in Seismic Zone 2, the design response spectrum for the site can be determined in accordance with *AASHTO 3.10.4*.

5.2.7 Seismic Performance Zones

Seismic performance zones for New Hampshire are determined based on the horizontal response spectral acceleration at 1.0-s period (SD1) in accordance with *AASHTO 3.10.6*:

- For all locations in the state with Site Classification A – D \Rightarrow Seismic Zone 1
Example: $S_{D1} = (F_{v \max D})(S_{1 \max}) = (2.4)(5.5\%) = 0.132 \leq 0.15$
- For locations with $S_1 \leq 4.29\%$ with Site Classification E \Rightarrow Seismic Zone 1
Example: $S_{D1} = (F_{vE})(S_1) = (3.5)(4.25\%) = 0.149 \leq 0.15$
- For all other locations in the state with Site Classification E \Rightarrow Seismic Zone 2
Example: $S_{D1} = (F_{vE})(S_{1 \max}) = (3.5)(5.5\%) = 0.193 > 0.15$

Note these values are determined using the maximum seismic factors for the state of NH. The designer should be using site-specific values in the equations to determine the seismic zone for each project site.

5.2.8 Calculation of Design Forces

The majority of New Hampshire bridges are in Seismic Zone 1 with low acceleration coefficients; in turn, earthquake forces will rarely govern the design. However, *AASHTO 3.10.9.2* requires that Seismic Zone 1 structures satisfy detailing requirements. These requirements pertain to the length of bearing seats supporting superstructure elements and the capacity of a force path for seismic loads to be transferred to substructure elements. Structures in Seismic Zone 2 shall be analyzed according to the minimum requirements specified in *AASHTO 3.10.9.3 and 4.7.4*.

For Seismic Zone 1, the nominal lateral force is determined as a percentage of the vertical reaction due to the tributary permanent dead load and tributary live loads assumed to exist during an earthquake (*AASHTO 3.10.9.2*). The minimum connection design forces are to be used in lieu of determining forces through a rigorous analysis.

- The AASHTO LRFD seismic design requirements indicate less seismic risk in the Northeast than what was shown in the LFD code.
- The load factor for live load, Extreme Event I, γ_{EQ} , shall be taken as 0.5 for all bridges as stated in Chapter 4, Section 4.3.5.
- For bridge designs in Seismic Zone 1, use a force of 25% of the tributary permanent vertical loads in accordance with *AASHTO 3.10.9.2*. Examination of the Peak Ground Acceleration Coefficient (PGA) Map for New Hampshire shows that all locations in the state yields a value of 25%.
- Seismic analysis is not required for single-span bridges, regardless of seismic zone.
- Connections between the bridge superstructure and the abutments shall be designed for the minimum force requirements as specified in *AASHTO 3.10.9*.
- Multi-span bridges shall follow the provision of *AASHTO 4.7.4.3*.
- Seismic effects for box culverts and buried structures need not be considered.

5.2.9 Minimum Support Length Requirements

The seat widths at supports in both the transverse and longitudinal directions shall be adequate to prevent a span from falling. The bridge seats at expansion bearings without restrainers, or dampers shall accommodate the greater of the maximum calculated seismic displacement (except for bridges in Seismic Zone 1) or a percentage of the minimum support length as specified in *AASHTO 4.7.4.4*.

- There are no seat width requirements if the bridge is integral.
- In the transverse direction, N is measured from the edges of piers or abutments to the centerline of the fascia beam. The *AASHTO Specifications* only provide requirements for the longitudinal seat widths. However, both directions shall be provided with adequate seat widths such that the superstructure has enough room to “ride out” an earthquake on its supports should the bearings and their connections fail.

5.2.10 Forces Transferred from Superstructure to Substructure

A. Seismic Zone 1

For Seismic Zone 1, minimum superstructure to substructure connection design forces are provided to prevent connections from becoming unintended weak links in the seismic lateral load path. Sufficient element strength and connectivity are needed to mobilize the lateral resistance of the main structural elements. Connections that transfer forces from one part of an element to another may include, but are not limited to, fixed bearings, expansion bearings with restrainer devices, and keeper blocks. A connection may be an element that simply restrains a member and may not physically connect to that member, such as keeper blocks. Anchorage detailing for connections should be extended far enough into the adjacent member to prevent premature failure and all superstructure components that are considered part of the load path to the connection (e.g., diaphragms), shall be designed for the minimum seismic design force. Girder support pedestals shall also be designed for the connection forces to prevent loss of span support.

1) Fixed Bearings

All fixed bearings, anchorage and pedestals shall be designed for a force of 25% of the tributary permanent vertical loads along with the tributary live loads in accordance with *AASHTO 3.10.9.2*.

2) Elastomeric Bearings

Elastomeric bearing vulcanized to masonry plate: The elastomeric bearing assembly shall be analyzed for a force of 25% of the tributary permanent vertical loads along with the tributary live loads in accordance with *AASHTO 3.10.9.2 and 14.7.5.3.7*. If the seismic force exceeds the elastomeric pad design, then a restraint element (e.g., keeper plates or keeper blocks) shall be designed for the seismic forces in excess of those accommodated by shear in the pad.

Elastomeric bearing resting on concrete, no masonry plate: There is no fully restrained direction due to flexibility of the bearings and there is no positive connection capable of transferring the force. Friction is not considered a positive connection due to uncertainty of vertical effects. The bearings will act like a “floating” bearing. The designer shall provide the minimum support length requirements for expansion bearings in the both the longitudinal and transverse directions.

3) Sliding Bearings (Seismic Zone 1)

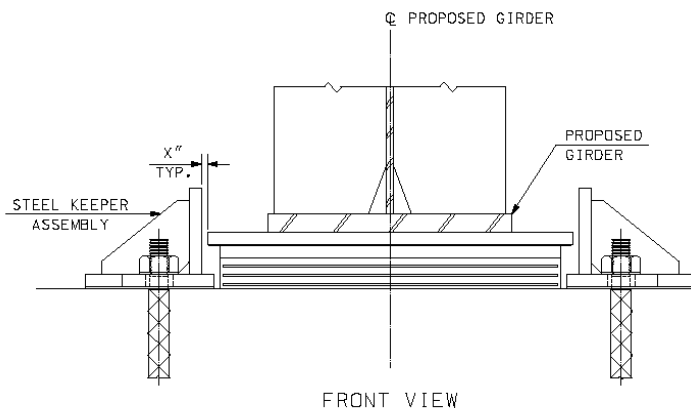
The minimum support length requirements for expansion bearings shall be provided in the longitudinal direction and restraint elements (e.g.; keeper plates or keeper blocks) shall be designed for a force of 25% of the tributary permanent vertical loads along with the tributary live loads in accordance with *AASHTO 3.10.9.2* in the transverse direction.

4) Pier Bearing Detail

All pier expansion bearing assemblies shall be detailed with keeper restraints (e.g., keeper plates or keeper blocks) for seismic movement in the transverse direction. The designer shall determine if keeper restraints will be located at each bearing or at the center and/or ends of the pier. See Figures 5.2.10-[1](#), [2](#) and [3](#) for examples.

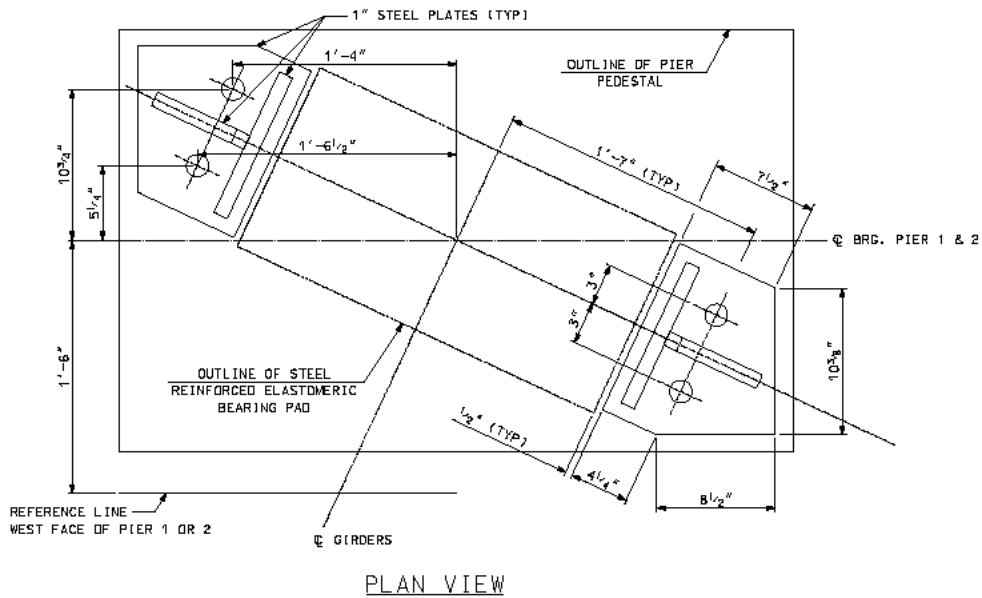
B. Seismic Zone 2

All connections and load path elements shall be designed for seismic forces by analysis in accordance *AASHTO 3.10 and AASHTO Guide Specifications for LRFD Seismic Bridge Design*.



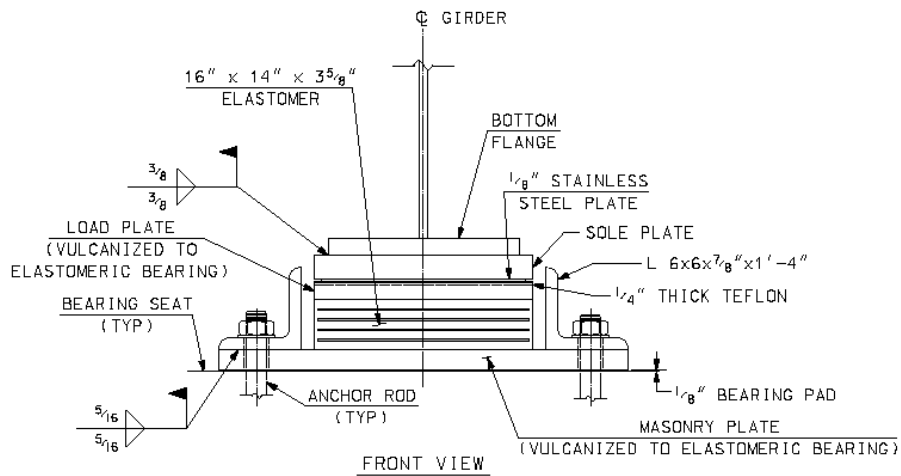
Elastomeric Bearing and Keeper Plate Assembly

Figure 5.2.10-1



Anchor Bolt Layout and Keeper Plate Assembly

Figure 5.2.10-2



Elastomeric Bearing and Keeper Angle Assembly

Figure 5.2.10-3

5.3 Rehabilitation of Existing Bridges

5.3.1 General

In general, retrofit work on piers and abutments will not be required. Most retrofit work shall consist of providing lateral restraint at bridge bearings and providing adequate seat width. The Department may require more extensive analysis and retrofit for major bridges, or if a unique situation exists. The decision to include seismic retrofit of piers and abutments will be made on a case-by-case basis by the Department. The level of seismic retrofitting that would be required for a particular rehabilitation or reconstruction project will be dependent on several factors such as ADT, importance of the bridge, age of the bridge, economic considerations, whether the bridge is on the Interstate, State, or Local System, etc.

When options for the designer are limited, providing adequate seat widths for an existing superstructure is a high priority over other potential retrofit measures; it is a key component of the Department's objective to prevent span loss. If attainment of adequate seat widths is not a practical option, using such measures as isolation bearings should be explored. To prevent significant transverse movement of the superstructure, concrete keeper blocks or steel keeper angles may be rigidly attached to the pier or abutment caps between the beams. When evaluating seismic retrofit options, all reasonable measures should be explored for possible implementation.

The following two FHWA reports should be used as guides for seismic retrofit of bridges:

- *Report No. FHWA-IP-87-6, "Seismic Design and Retrofit Manual for Highway Bridges"*
- *Report No. FHWA-RD-94-052, "Seismic Retrofitting Manual for Highway Bridges"*

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5.4 Seismic Isolation Bearings

5.4.1 General

Seismic design incorporating seismic isolation devices may be considered for bridges classified as critical, provided that the following requirements are satisfied:

- The design of the seismic isolation devices is in accordance with *AASHTO Guide Specifications for Seismic Isolation Design*, as modified in this Manual. The response modification factor for all elements shall be taken as 1.0.
- It can be demonstrated that the performance characteristics of the bearings will provide a necessary benefit for the design of the bridge substructures.
- The specific isolation systems and manufacturers being considered shall meet the approval of the Design Chief.
- At the preliminary and final design stages, the design of the isolators must be closely coordinated with all isolation device manufacturers approved by the Design Chief.
- Non-seismic loads and movements are determined and adequately accounted for in the design of the isolation systems(s).
- The larger displacement resulting from an increase in the period of vibration due to increased flexibility is adequately accommodated and detailed (such as providing adequate roadway joint movement capability or designing and detailing a portion of the backwall to break away upon superstructure impact).
- Design and cost information shall be obtained for a cost evaluation of the various seismic design alternatives.
- Life-cycle cost of additional and specialized bearing inspections shall be calculated.
- Issues related to long-term performance and maintenance shall be identified.

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5.5 Seismic Design Requirements for Retaining Walls

5.5.1 General

The seismic design for retaining walls shall be determined in accordance with the *AASHTO 11.5.4.2 and 11.6.5* as noted below:

- If the wall is located in Seismic Performance Zones 1-3 and meets the criteria as noted in *AASHTO 11.5.4.2*, a no-seismic analysis option can be taken for the internal and external seismic stability design of the wall.
- Seismic analysis shall be considered for walls greater than 30-ft. (9.1-m) and located in Seismic Zones 2 or higher as described in *AASHTO C11.5.4.2*.

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References

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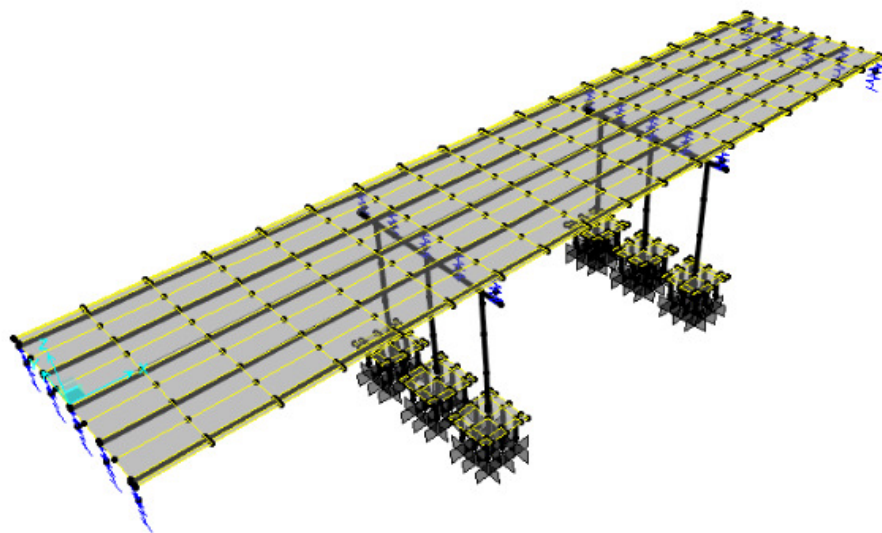
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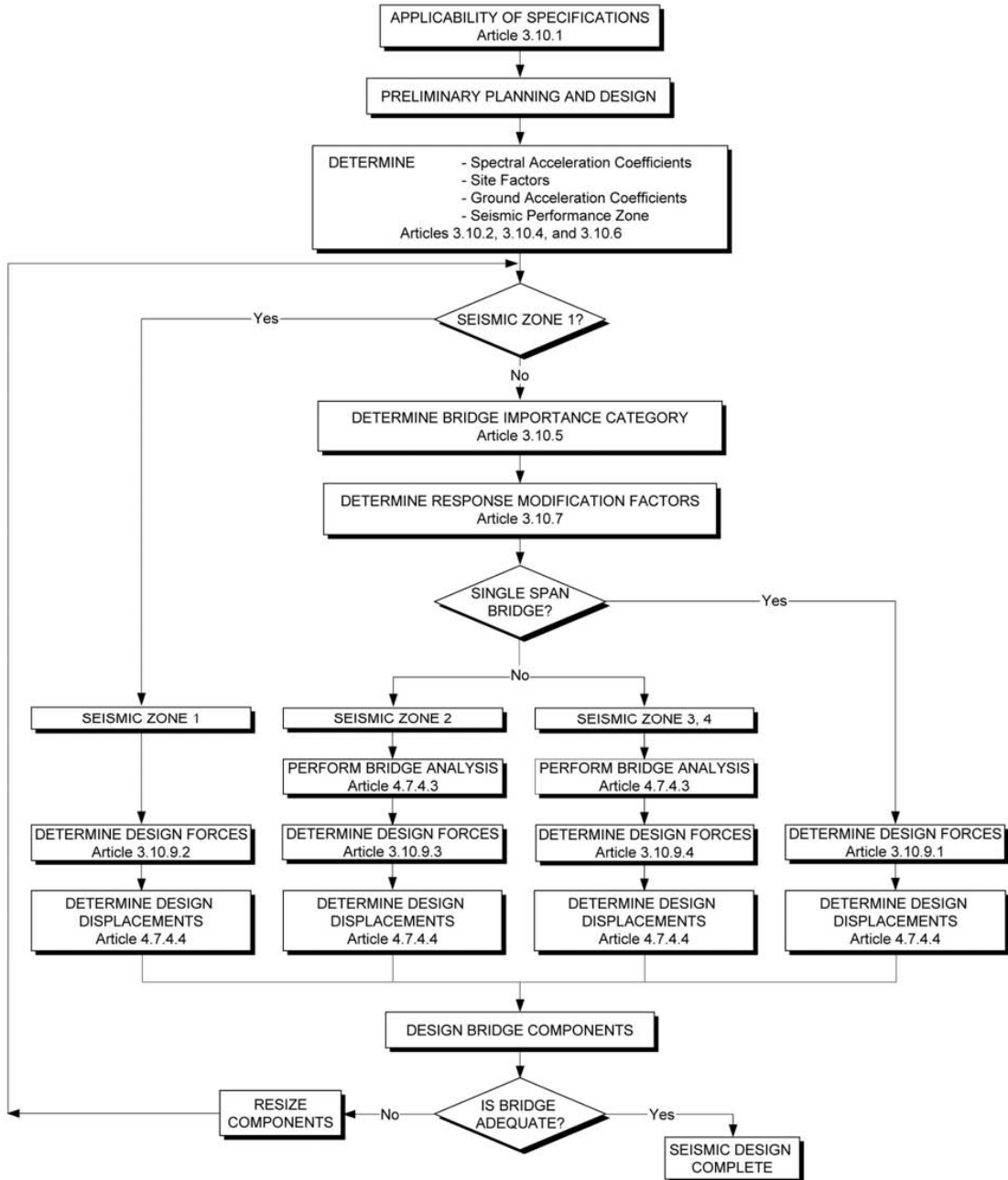
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Reference: AASHTO LRFD, Section 3, Appendix A3

APPENDIX A3—SEISMIC DESIGN FLOWCHARTS



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