9.0 CONCLUSIONS

9.1 Summary

The primary objectives of this study were twofold. Objective 1 was to establish a current subsurface and earthquake design geographic information systems (GIS) database for the counties of Butler, Stoddard, New Madrid, Franklin and St. Louis. Objective 2 was to conduct detailed earthquake assessments at two sites along designated emergency vehicle priority access route US 60.

9.2 Geotechnical GIS Databases

Databases have been established for earthquake design data for the US 60 corridor in Butler, Stoddard and New Madrid Counties and for the MO 100 corridor in Franklin and Saint Louis Counties. This includes appropriate data from Missouri Department of Transportation files. These databases will be integrated into the existing Missouri Department of Transportation GIS system for future access, and serve as the beginning of a larger regional or statewide database.

For future development and usage by Missouri Department of Transportation. Further details and access procedures may be found in "User Instructions for Data Entry and Editing-Database of Borehole and Other Geotechnical Data for Missouri Highway Structures".

9.3 Site Specific Earthquake Hazards Assessments

Detailed earthquake site assessments were conducted for two critical US 60 roadway sites (Wahite Ditch Site and St. Francis River Site). Site assessments included: subsurface exploration, and laboratory testing to identify subsurface materials and their engineering properties; evaluation of available seismic records and procedures to characterize the ground motions associated with various design earthquake events; and evaluation of the response of the subsurface materials and the existing bridge structures to the estimated ground motions.

The goals of the site assessments at these two locations were to:

1. Estimate peak magnitude and duration of ground surface motion (including amplification/damping) associated with various events at each site.

2. Evaluate the susceptibility of each site to quake-induced slope instability, liquefaction and flooding.

3. Estimate shaking effects on the various types of existing bridge structures at each site.

4. Compare ground motion and structural response parameters from site-specific earthquake analysis method with those from AASHTO response spectrum analysis method and provide preliminary guidance regarding selection of the analysis method at future sites.

5. Evaluate modified site assessment techniques and establish a basis for using these modified techniques at other sites along designated emergency access routes.

Site-specific seismic response evaluations or the four study bridges were completed. Liquefaction potential, slope stability, abutment stability, flooding potential, and structure stability analysis were performed at both sites for selected “worst case scenario bedrock ground motions” with PE of exceedance of 2% and 10% in 50 year, respectively. Ground motion analysis utilized synthetic ground motions for a New Madrid and other, source zones. Results are presented in Section 8.

9.3.1 St. Francis River Site

The following conclusions may be drawn from this study:

9.3.1.1 Liquefaction

The soil does not liquefy under selected ground motion for PE 10 in 50 years. However, the soil at this site liquefied for PE 2% in 50 years to different depths depending on the magnitude and the factor of safety.

9.3.1.2 Slope Stability

The abutment slopes at the St. Francis River Bridge site are stable under all but the most extreme earthquake events.

9.3.1.3 Flood Hazard

Approximately 5.7 miles of US 60 roadway, from the St. Francis River eastward to approximately 0.4 miles west of Highway WW/TT (which leads to Dudley), and 3.4 miles of roadway from approximately 0.3 miles east of Highway WW/TT eastward to Highway ZZ would flood during and after an earthquake event that resulted in the failure of Lake Wappapelo Dam. Several additional stretches of roadway could flood as a result of levee failures.

9.3.1.4 Structural Response of St. Francis River Bridges

9.3.1.4.1 New St. Francis River Bridge

The three-span bridge with integral abutments was analyzed and evaluated in detail under the excitation of 12 ground motions. The overall performance of the bridge is satisfactory except for the following observations. It was found that the steel plates of the neoprene elastomeric pads are not anchored into the capbeam with the required embedment length. They may be pulled out during earthquakes. The diagonal members of the diaphragms or cross-frames are vulnerable to the 2% PE earthquakes. Comparison between the response spectrum analysis and the time history analysis verified the sufficient adequacy of analyzing the bridge with integral abutments using the simpler response spectrum procedure.

9.3.1.4.2 Old St. Francis River Bridge

Based on the extensive analysis and detailed evaluation of the bridge, the following conclusions can be drawn. The support length of bearings is slightly short based on the current requirement, which may result in the dropping of the spans adjacent to expansion joints. The shear capacity of anchor bolts and the embedment length of bearings are also inadequate for both 10% and 2% likelihood earthquakes. Another major concern is the stability of columns. Although the C/D ratios with a ductility indicator of 5 is greater than 1.0 for all columns, they are likely insufficient to sustain large deformations due to the poor detailing at joints. Associated with the poor detailing is a greater concern for shear capacity of the columns. Just like the New St. Francis River Bridge, it is likely that the diagonal members of the diaphragms and cross frames of this bridge would buckle during a strong earthquake event.

It is also observed from analyses that the response spectrum method can give internal shears and moments as well as displacements with satisfactory engineering accuracy for linear bridges with seat-type abutments. However, potential pounding at expansion joints during a strong earthquake event makes the spectrum method invalid.

9.3.1.4.3 Old St. Francis River Bridge Abutment

The maximum displacement at the top of this abutment varied from 0.43 inch to 1.02 inch for 10% PE and 2% PE for the different magnitudes of earthquakes. This displacement is tolerable without any damage to the abutments.

9.3.2 Wahite Ditch Site

9.3.2.1 Liquefaction

The soils do not appear to liquefy for the 10% PE and M 6.4. However, the soils do liquefy marginally for a 10% PE earthquake with a magnitude of 7.0.

For a 2% PE and M 7.8 and 8.0 earthquake with factors of safety less than 1.0, the soils liquefy throughout.

9.3.2.2 Slope Stability

This site is expected to be stable under small earthquake conditions. The site is less sensitive to ambient ground-water levels (which are affected by water levels in the river) than at the St. Francis River site. Stability analysis under large earthquake conditions indicates marginal stability at the Wahite Ditch site when ground-water levels are high.

9.3.2.3 Flood Hazard

Water levels appear to be too low during normal conditions to pose a significant risk of exiting the channel, even in the event of levee failure. Furthermore, the roadway is elevated above the surrounding land. One section of roadway located 0.1 to 0.5 miles west of the ditch is at low elevation and could potentially flood.

9.3.2.4 Structure Stability of Wahite Ditch Bridges

9.3.2.4.1 New Wahite Ditch Bridge

Extensive analyses and detailed evaluation of the bridge indicated that the bridge can sustain an earthquake at both 10% and 2% probability of exceedance. The only components that warrant attention are the shear keys on top of the capbeam. Their capacity is slightly inadequate for two out of the six earthquakes at high hazard levels.

9.3.2.4.2 Old Wahite Ditch Bridge

Based on the extensive analyses and evaluations of the bridge, some conclusions can be drawn as follows. The support length of the superstructures is insufficient so that it is likely that the bridge deck will drop off its support at expansion joints. Other load transferring components, such as bolts and their embedment lengths and edge distances, are also inadequate for earthquake loads. Even though a ductility indicator of 5 was used, the C/D ratios of the columns at Bent 3 are still less than one, indicating insufficient strength. Associated with the column bending, the shear capacity of the columns is significantly less than required due to the lack of transverse reinforcement. Like the diaphragms and cross frames built with angles at the St. Francis River Bridge site, the diagonal members are vulnerable to buckling.

9.3.2.4.3 Old Wahite Ditch Bridge Abutment

The maximum displacement at the top of this abutment for a 10% PE, M 6.4 and a 2% PE and M 8.0 earthquake varies from 0.28 inch to 0.47 inch. This displacement will not cause any damage to the abutments.

10.0 RECOMMENDATIONS

10.1 Protocol

Earthquake hazard assessment at both the St. Francis River and the Wahite Ditch Sites was essentially a six-component process consisting of the following inter-related analyses:

1) Determination of the appropriate earthquake induced strong ground motion.
2) Determination of liquefaction potential in response to strong ground motion.
3) Determination of slope stability in response to strong ground motion.
4) Evaluation of abutment stability in response to strong ground motion.
5) Evaluation of structure stability in response to strong ground motion.
6) Determination of potential for flooding in response to strong ground motion.

Based on this study, the following recommendations are made with respect to the development of an effective protocol for conducting these six analyzes.

10.1.1 Determination of Site-Specific Strong Rock Motion

Recommendation 1: Acquire/develop capability to generate site-specific synthetic ground motion at bedrock based on earthquake magnitude, source signature (amplitude and phase spectrum), and source distance/depth.

Recommendation 2: Arbitrarily select two source zones, one proximal and one distal, based on recommendations from The Missouri Department of Natural Resources geologists. The Commerce Geophysical Lineament could serve as a reasonable proximal source zone for further studies along US 60. The New Madrid Fault zone could serve as a reasonable distal source zone.

Recommendation 3: Generate a representative suite of synthetic bedrock ground motions for both the proximal and distal sources. The representative suite of synthetic ground motions should cover a range of potentially damaging magnitudes (perhaps 4 to 8), and vary in duration and frequency.

Recommendation 4: Propagate the suite of bed rock synthetic ground motions to the surface, and access damage as per analysis 2 through 5 (section 8.1).

Recommendation 5: Estimate probability of occurrence of each ground motion, based on input from USGS and other sources.

Summary: The procedure outlined above could be of more long-term utility to Missouri Department of Transportation than the "worst case New Madrid source zone scenario" process employed in this study. Earthquake probability estimates and prospective source zone locations are likely to change over time (in response to new data) – more so than the generated suite of synthetic ground motions. If this assumption is correct, Missouri Department of Transportation would merely be able to reassign new probabilities to each synthesized outcome – as opposed to having to generate new synthetic ground motions in response to changing probabilities.

10.1.2 Determination of Liquefaction Potential

Recommendation 1: Liquefaction analysis should be conducted (using the entire suite of synthetic ground motions) at locations of all critical roadway structures and in roadway areas where there is a paucity of structures (to ensure valid statistical sampling). Areas along the roadways should be designated in accordance with their propensity for liquefaction (re: magnitude and source distance). Probabilities can be assigned thereafter, and reassigned as probability estimates change over time.

Recommendation 2: Seismic cone penetrometer data should be acquired to a depth of 50 feet (if possible) in immediate proximity to structures studied. Soil should be sampled from surface to bedrock, and SPT data should be acquired to enable the development of a vertical soil profile.

Recommendation 3: Bedrock ground motion should be propagated to surface. The propensity of each soil layer to liquefy under synthesized ground motion should be determined.

10.1.3 Determination of Slope Stability

Recommendation 1: Slope stability analysis should be conducted (using the entire suite of synthetic ground motions) at all critical roadway bridge sites and in selected roadway areas where there may be some potential for lateral spreading. The sites studied should be designated in accordance with their propensity for slope failure (re: magnitude and source distance). Probabilities can be assigned thereafter, and reassigned as probability estimates change over time.

Recommendation 2: At each site, topographic data and shallow subsurface control (engineering properties of soil) should be acquired (trenching and boreholes).

Recommendation 3: Bedrock ground motion should be propagated to surface. The propensity of the slope to fail under synthesized ground motion should be determined.

10.1.4 Determination of Potential for Flooding in Response to Strong Ground Motion

For future analysis of earthquake-induced flooding of roadway sections, we recommend the following procedure:

1. Preliminary identification of regions susceptible to flooding:
    - Collect report information on anticipated flood run out following catastrophic failure of nearby dams.
    - Collect 7.5-minute topographic maps and FEMA flood hazard maps along the alignment under evaluation.
    - Identify river, creek, and drainage ditch locations, approximate elevations of water levels, and approximate elevations of both natural and man-made levees flanking the waterways.
    - On the topographic maps, subdivide zones along the roadway by 5-foot contour intervals.
    - Mark areas where the land is below water levels in waterways as zones of potential flooding.
    - Field check each area to visually assess the elevation of the roadway compared to surrounding land.
2. Specific assessment of regions identified as susceptible:
    - Assemble more accurate data on range of water elevations in canals and streams through contact with local government offices for agriculture, flood control, and public works.
    - Measure ranges of water elevations in canals and streams using GPS devices.
    - Develop more accurate topographic analysis using DEM computer files analyzed with GIS software to compare water levels and roadway elevations.

- Confirm computer topographic analysis with GPS field measurements of roadway elevations.
- Use DEM computer files, soil survey information, and field reconnaissance data on soil type and strength to rank the susceptibility to slope failure of various stretches of waterway levees. Combine this analysis with roadway and water elevation analysis to identify critical areas where likelihood of levee failure is high and flooding potential in the event of levee failure is also high.

10.1.5 Evaluation of Flooding Potential

Flood analysis (in accordance with methodologies outlined in this report) should be conducted for the entire designated roadway. Additionally, slope stability analysis (as outlined above) should be conducted at selected sites along river, drainage ditch and irrigation canals to determine likelihood of failure that could result in flow blockage and flooding.

10.1.6 Determination of Structural Stability

10.1.6.1 Evaluation of Abutment Stability

Recommendation 1: Abutment stability analysis should be conducted (using the entire suite of synthetic ground motions) at all critical roadway structures. Sites studied should be designated in accordance with their propensity for abutment failure (re: magnitude and source distance). Probabilities can be assigned thereafter, and reassigned as probability estimates change over time.

Recommendation 2: Bedrock ground motion should be propagated to surface. The integrity of each abutment under synthesized ground motion should be determined.

10.1.6.2 Evaluation of Stability of Integrated Bridge Abutments

Recommendation 3: Structure stability analysis should be conducted (using entire suite of synthetic ground motions) for all critical roadway structures. Sites studied should be designated in accordance with their propensity to fail (re: magnitude and source distance). Probabilities can be assigned thereafter, and reassigned as probability estimates change over time.

Recommendation 4: Bedrock ground motion should be propagated to surface. The propensity of each designated structure to fail under synthesized ground motion should be determined.

10.1.6.3 Evaluation of Stability of Structural Members

Recommendation 5: For multiple span highway bridges with integral abutments, the response spectrum analysis is accurate enough for evaluation of structural members with a linear bridge model. For bridges supported by seat-type abutments at their ends, pounding at expansion joints makes it necessary to analyze a geometrically nonlinear system of the bridges.

Recommendation 6: For bridge seat-type abutments, the load transferring members such as bolts and their anchorage and edge distances must be evaluated together with the minimum support length requirements. For existing bridges, the shear and moment capacities of the columns must be evaluated with considerations of the detailing at the beam to column and the column to footing joints.

10.2 Further Work

This study has provided a sound basis for developing a comprehensive evaluation of seismic response of highway structures in southeast Missouri. Based on these results and on discussions with Missouri Department of Transportation personnel the following recommendations are made for further work.

10.2.1 Proposed Study: Retrofit of Critical Structures along Designated Emergency Vehicle Priority Access Routes

The results of this study have identified a number of critical locations where the bridge and embankment structures would fail under the severe earthquake loading forecasted for this area. Consequently, since these faculties must meet emergency access serviceability, it is proposed to develop seismic retrofit procedures for enhancing the ability of these structures to resist the severe earthquake forces. The procedures could include structural stiffening of the bridge members, enhanced resistance to embankment and foundation liquefaction and slope failure. This research should also include the development of a post-earthquake evaluation protocol such that the critical structures could be quickly and easily evaluated to determine their structural integrity following the earthquake event.

10.2.2 Proposed Study: Site Specific Earthquake Assessments along MO 100

The Missouri Department of Transportation in conjunction with other state agencies has designated specific routes for vehicular access of emergency personnel, equipment and supplies in the event of a major earthquake event in southeast Missouri. These routes include portions of MO 100, US 50 and I-44. The routes traverse varied geologic settings and include or cross many critical roadway features such as bridges, slopes, box culverts, and retaining walls. The extent of damage and survivability of these critical roadway features in the event of a major earthquake event is not fully known and would impact the ability to use these designated routes to provide emergency vehicular access in a timely manner.

The goals of this proposed study are to use the results of this Phase I US 60 study to complete a regional overview and prioritization of seismic hazards and to conduct site specific studies along the next critical highway, which is judged to be MO 100. The specific objectives to complete these goals are given below. Detailed earthquake assessments will be conducted for two sites where critical roadway features exist. These site assessments will include subsurface exploration and laboratory testing to identify subsurface materials and their engineering properties; evaluation of available seismic records and procedures to characterize the ground motions associated with various design earthquake events and evaluation of the response of the subsurface materials and the existing bridge structures to the estimated ground motions. Site assessment techniques will be selected based on their usefulness as determined from this study. In this way, comparisons in data quality, investigation time, and investigation costs may be made between the detailed US 60 study and the more streamlined MO 100 study.

It is proposed that members of the research team will survey MO 100 in St. Louis and Franklin Counties. Sites with critical roadway features will be visually evaluated and ranked based upon geologic factors, structural factors and perceived criticality/risk factors. The top two sites with differing geologic settings will then be selected for further study.

The goals of the site assessments at these locations would be to:

1. Estimate peak magnitude and duration of ground surface motion (including amplification/damping) associated with various events at each site.

2. Evaluate the susceptibility of each site to earthquake induced slope instability and liquefaction.

3. Estimate shaking effects on the various types of existing bridge structures at each site.

4. Compare ground motion and structural response parameters from site-specific earthquake analysis method with those from AASHTO response spectrum analysis method and provides preliminary guidance regarding selection of the analysis method at future sites.

5. Evaluate the modified site assessment techniques identified in the US 60 study and establish a basis for using these modified techniques at other sites along designated emergency access routes.

Finally, a qualitative assessment of slope stability along the entire length of MO 100 from near Linn to Manchester will be completed, as well as an assessment of evidence of previous earthquake activity (in the form of sand blows, prehistoric slope movement, etc.).

10.2.3 Proposed Study: Regional Liquefaction Hazard Analysis

Liquefaction hazards will be identified and prioritized along the designated emergency vehicle routes US 60 and MO 100 using information in the GIS database prepared for Phase I and future work. Strip maps showing liquefaction potential along designated routes will be generated.

10.2.4 Proposed Study: Geo-Referencing of Boring Locations

The locations of geotechnical borings at the two bridges evaluated in Phase I and at the Phase II sites will be precisely identified in the GIS database using as-built drawings, survey information, and geo-referencing software. This will allow accurate cross-section generation from the database information without a field visit. Boring locations in the current GIS database are limited to station and offset coordinates, which are not precise enough for cross-section and mapping applications. Additionally, plans from approximately 104 bridges (4 plans per bridge) will be scanned and geo-referenced to permit accurate locating of proximal boreholes.

10.2.5 Proposed Study: Regional Prioritization for Future Earthquake Hazards Assessments

Part of the US 60 study included development of a GIS database of subsurface and earthquake data for both the US 60 and MO 100 corridors. This study will couple an assessment of this database with a regional review of geologic, hydrologic, and road structure information to prioritize future earthquake assessments along MO 100, US 50 and I-44. This type of assessment was completed for the Phase I study along US 60, and the methodology will be revised and conducted in more detail for the proposed Phase II study. The results this assessment are expected to provide the basis for gauging the sensitivity of various roadway and geologic conditions to earthquake damage and prioritizing locations for further study. In addition to bridges and roadway conditions, the assessment will also qualitatively evaluate slope stability hazards and flooding hazards related to levee, dam, or canal failure.

10.2.6 Proposed Study: Laboratory Testing of Truss-Type Diaphragms or Cross Frames and Effective Retrofitting Techniques

Three out of four bridges investigated in this project have diaphragms or cross frames consisting of angles. They are all subject to high potential for buckling during strong earthquakes. To ensure that the superstructure (deck, girder, diaphragms/cross frames) remains integrated to transfer load from it to the substructure, the diaphragms need to be further studied for the development of practical retrofitting techniques.

10.2.7 Proposed Study: Integration of LOGMAIN Surficial Material Information

Database elements will be identified to permit surficial materials information in LOGMAIN to be integrated into the Missouri Department of Transportation database.

10.2.8 Proposed Study: Long Term Strategic Plan

The site-specific and regional studies will be used to develop a long term strategic plan for earthquake hazards assessment in Southeast Missouri. The strategic plan will contain two elements: the first will be a prioritization of structures or sections of highway for further study of specific seismic hazards (shaking, slope movement, flooding, liquefaction, etc.), and the second will be a plan for solicitation of continued funding for continued funding of additional phases of the project. The primary agencies or programs targeted will be FEMA, USGS, NEHRP, and NSF.

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12.0 LIST OF SYMBOLS

Symbol	**Definition**
A	Cross section of single pile
A_{bolt}	Cross-sectional area of one bolt
$A_{b,splice}$	Area of spliced bar
A_g	Gross area of column cross-section
A_s	Area of steel in cross-section
A_{tr}	Area of transverse steel reinforcement
$A_{tr(c)}$	Capacity for transverse confinement
$A_{tr(d)}$	Demand for transverse confinement
$a_{(PGHA)}$	Peak horizontal ground acceleration
$a_{(PGVA)}$	Peak vertical ground acceleration
$acc_x(t)$	Horizontal ground acceleration
$acc_v(t)$	Vertical ground acceleration
B	Footing (pile cap) width, bridge abutment width
b	Width of cross-section
c	Cohesion (psf)
CPT	Cone penetrometer test
CRR	Cyclic resistant ratio
CSR	Cyclic stress ratio
c_x	Damping of single pile for translation along x axis
c_x^g	Damping of piles group for translation along x axis
$c_{x\phi}$	Cross coupled damping of single pile for coupling sliding along x-axis
$c_{x\phi}^{g}$	Cross coupled damping of piles group for sliding along x-axis and
c_y	Damping of single pile for translation along y axis
c_y^g	Damping of piles group for translation along y axis
$c_{y\theta}$	Cross coupled damping of single pile for sliding along y-axis and
$c_{y\theta}^{g}$	Cross coupled damping of piles group for sliding along y-axis and
c_z	Damping of single pile for translation along z axis
c_z^g	Damping of piles group for translation along z axis
c_ϕ	Damping of single pile for rocking about y axis
c_ϕ^{g}	Damping of piles group for rocking about y axis
c_θ	Damping of single pile for rocking about x axis
c_θ^g	Damping of piles group for rocking about x axis
c_ψ	Damping of single pile for torsion about z axis
c_ψ^{g}	Damping of piles group for torsion about z axis
d	Effective width of cross-section
d_b	Diameter of reinforcing bar
d_{bolt}	Diameter of one bolt
D	Pile diameter
D_r	Relative density (unitless)
e	Void ratio (unitless)
E_p, E_{pile}	Modulus of elasticity of pile material
F_u	Ultimate stress for one bolt

F_v	Shear stress for one bolt
f'_c	Compressive strength of concrete
f_s	Stress in steel
f_y	Steel yield stress
f_{yt}	Transverse steel yield stress
f_{w1}, f_{T1},f_{x1},$f_{\phi1}$, $f_{x\phi1}$	Stiffness parameters
f_{w2}, f_{T2}, f_{x2}, $f_{\phi2}$, $f_{x\phi2}$	Damping parameter
G_s	Specific gravity of soil particles (unitless)
g	Acceleration due to gravity
Go	Initial shear modulus of soil
Gs	Shear modulus of soil
H	Abutment height
H_1	Horizontal seismic force increment due to weight of abutment
H_2	Horizontal force increment as result of weight of girder and traffic load
H_3	Horizontal seismic force due to soil mass above wall
HGA	Horizontal ground acceleration (% of gravity)
Ip	Moment of inertia of single pile about x or y axis
Ip_p	Polar moment of inertia of single pile
j	Parameter for concrete stress distribution
k_1	FHWA parameter for transverse confinement
k_2	FHWA parameter for transverse confinement
k_3	Effectiveness of transverse bar anchorage
k_h, k_v	Horizontal and vertical seismic coefficient, $k_h = acc_x(t)/g$ and $k_v = acc_y(t)/g$
k_m	FHWA parameter for longitudinal reinforcement
k_x	Stiffness of single pile for translation along x axis
$k_x{}^g$	Stiffness of group of piles in translation along x axis
$k_{x\phi}$	Cross coupled stiffness of single pile for coupling along x-axis and rocking about y axis
$k_{x\phi}{}^g$	Cross coupled stiffness of piles group for sliding along x-axis and rocking about y axis
k_y	Stiffness of single pile for translation along y axis
$k_y{}^g$	Stiffness of piles group for translation along y axis
$k_{y\theta}$	Cross coupled stiffness of single pile for sliding along y-axis and rocking about x axis
$k_{y\theta}{}^g$	Cross coupled stiffness of piles group for sliding along y-axis and rocking about x axis
k_z	Stiffness of single pile for translation along z axis
$k_z{}^g$	Stiffness of piles group for translation along z axis
k_ϕ	Stiffness of single pile for rocking about y axis
$k_\phi{}^g$	Stiffness of piles group for rocking about y axis
k_θ	Stiffness of single pile for rocking about x axis
$k_\theta{}^g$	Stiffness of piles group for rocking about x axis
k_ψ	Stiffness of single pile for torsional about z-axis
$k_\psi{}^g$	Stiffness of piles group for torsional about z-axis
L	Pile length

$l_{a(c)}$	Capacity of longitudinal anchorage
$l_{a(d)}$	Demand of longitudinal anchorage
L_{col}	Length of column
l_s	Length of splice
M	Magnitude
m	Mass of bridge abutment
Mm	Mass moment of inertia of bridge abutment about the axis of rotation
M_ϕ	Moment about y-axis.
M_θ	Moment about x-axis
N_1	Field measured standard penetration value (number of blows per foot)
$N_{1,60}$	Corrected standard penetration value (number of blows per foot)
Nspt	Standard penetration value (number of blows per foot)
P	Axial compressive force
Pa, ΔPae	Static and dynamic increment of earth pressure
Px ,Py	Total horizontal force in x or y direction
PHGA	Peak horizontal ground acceleration (% of gravity)
PVGA	Peak vertical ground acceleration (% of gravity)
Q	Vertical force acting on the top of bridge abutment transmitted from girder
$r_{ad\text{-}long}$	C/D ratio for longitudinal abutment displacement
$r_{ad\text{-}trans}$	C/D ratio for transverse abutment displacement
$r_{bf\text{-}edge\ dist.}$	C/D ratio for edge distance of bolts
$r_{bf\text{-}embed}$	C/D ratio for bolt embedment length
$r_{bf\text{-}embed\text{-}adj}$	C/D ratio for bolt embedment length, adjusted for stresses
$r_{bf\text{-}long}$	C/D ratio for shear in longitudinal direction
$r_{bf\text{-}trans}$	C/D ratio for shear in transverse direction
$r_{ca\text{-}bottom}$	C/D ratio for reinforcement anchorage at column bottom
$r_{ca\text{-}top}$	C/D ratio for reinforcement anchorage at column top
r_{cc}	C/D ratio for transverse confinement
r_{cross}	C/D ratio for diaphragm and cross-frame members
r_{cs}	C/D ratio for splices
$r_{cs\text{-}adj}$	C/D ratio for splices, adjusted for steel stresses
r_{cv}	C/D ratio for column shear
r_{ec}	C/D ratio for column moment
r_o	Pile radius
S	Spacing (c/c distance of piles in all directions), saturation of soil (unitless)
s	Spacing of transverse reinforcement
SPT	Standard penetration test
T	Torsional moment
V_1	Vertical seismic force increment due to weight of abutment
V_2	Vertical force increment as result of weight of girder and traffic load
V_3	Vertical seismic force due to soil mass above wall
$V_{b(c)long}$	Shear capacity in longitudinal direction
$V_{b(d)long}$	Shear demand in longitudinal direction
$V_{b(c)trans}$	Shear capacity in transverse direction
$V_{b(d)trans}$	Shear demand in transverse direction

$V_{e(d)}$	Elastic shear demand in columns
$V_{i(c)}$	Initial shear capacity of column (concrete & steel)
$V_{f(c)}$	Final shear capacity of column
VGA	Vertical ground acceleration (% of gravity)
V_p	Shear wave velocity of pile material
V_s	Shear wave velocity of soil
$V_{u(d)}$	Maximum column shear from plastic hinging
v_c	Initial shear capacity of concrete in column
W	Weight of bridge abutment
Ws	Weight of soil above of bridge abutment
X	Translation along x axis
x	Axis perpendicular to abutment and pier of bridge (direction of traffic), distance in x-direction
x_r	Distance between C.G. of footing (pile cap) and center to center of a pile
Y	Translation along y axis
y	Axis parallel to abutment and pier, distance in y-direction
Z	Translation along z-axis
z	Axis in vertical direction, distance in z-direction
z_c	Distance between center of gravity and base of footing (pile cap)
α_h	Horizontal interaction factor
α_A	Vertical interaction factor
α_{Lx}, α_{Ly}	Horizontal interaction factor in x and y direction
β	Departure angle
δ	friction angle at interface of soil and wall
γ_{water}	Unit weight of water (pcf)
γ_{dry}	Dry unit weight of soil (pcf)
θ	Footing rotation
μ	FHWA multiplier for transverse confinement
φ	Multiplier for shear capacity for bolts
φ	Internal friction angle of soil, rotation about y axis
ν_p	Poisson ratio of pile material
ν_s	Poisson ratio of soil
$\rho(c)$	Volumetric ratio of existing transverse reinforcement
$\rho(d)$	Required volumetric ratio of transverse reinforcement
ρ_p	Mass density of pile material
ρ_s	Mass density of soil
σ	Total vertical stress
σ_o'	Effective initial vertical stress
ψ	Rotation about z axis
θ	Rotation about x axis
τ	Total shear stress