Appendix C   Basis of Design

COWI

City of Kirkland

Totem Lake Non-Motorized Bridge - Basis of Design

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Amendments Record Sheet

Executive Summary

The document prescribes the design standards and methodology to be used in the design of the Totem Lake Connector Bridge project. It provides an explanation of the design approach and lists the hierarchy of relevant codes and guidelines.

Key topics included in this document include: Bridge Geometry, Design Loads, Deflection Criteria, Vibration Criteria, Geotechnical Considerations, and Analysis.

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Totem Lake Non-Motorized Bridge - Basis
of Design

1 Introduction

The following criteria are proposed for the design of the non-motorized bridge to be constructed to carry the Cross Kirkland Corridor (CKC) trail over NE 124th St. and 124th Ave NE (Totem Lake Blvd NE). This Basis of Design document is subject to revision and additions as preliminary design proceeds and input is received.

1.1 Design Codes

The standards to be used in the bridge design are listed below in order of precedence. This encompasses the design of the bridge control line, superstructure, substructure, foundations, abutments, railings, bearings, seismic performance, and vibration criteria.

i.   AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, 2009 (AASHTO Ped)

ii. Service d'Etudes Techniques des Routes et Autoroutes, Footbridges – Assessment of Vibrational Behaviour of Footbridges under Pedestrian Loading – Practical Guidelines, 2006 (SETRA)

iii. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (AASHTO Seismic)

iv. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (AASHTO LRFD)

v.   AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, First Edition, 2015 (AASHTO Signs)

vi. Washington State Department of Transportation Bridge Design Manual (LRFD), 2016 (BDM)

vii. American Institute of Steel Construction, Steel Construction Manual, 14th Edition, 2011 (AISC)

viii. American Concrete Institute 318-14, Building Code Requirements for Structural Concrete, 2014 (ACI)

ix. AASHTO Guide for the Development of Bicycle Facilities, 4th Edition, 2012 (AASHTO Bike)

x.   ADA Standards for Accessible Design, 2010 (ADA)

xi. National Association of City Transportation Officials Urban Bikeway Design Guide – 2nd Edition, 2014 (NACTO)

Exceptions:

- AASHTO Ped states in Section 1.1 - Scope that "Pedestrian bridges with cable supports or atypical structural systems are not specifically addressed". It may be the case that several provisions in this code are excessively conservative or unachievable for bridges of this type, especially regarding deflections and vibrations. In these cases, in lieu of AASHTO Ped, it is intended to use the design approach laid out in SETRA.

- AASHTO Seismic states in Section 3.1 that the guide specifications "shall be taken to apply to the design and construction of conventional bridges… For non-conventional bridges, the Owner shall specify appropriate provisions, approve them, or both."

1.2   Design Life

The design life shall be 75 years, consistent with AASHTO LRFD Section 1.2.

1.3   Materials

Bridge piers shall be comprised of:

- Painted steel, weathering steel, or reinforced concrete.

Within the roadway clear zone, substructure elements shall be capable of withstanding vehicle collision loads unless protected by roadside barriers.

Bridge superstructure material considerations shall include:

- Painted steel, weathering steel, and/or reinforced concrete.

Miscellaneous elements shall be stainless steel, galvanized steel, aluminum, or wood.

Galvanized steel over the wetlands is not allowed.

Railings shall be comprised of:

- Painted steel, galvanized steel, stainless steel, and/or aluminum.

Bearings shall be elastomeric, lead core elastomeric, pendulum, or disc bearings.

2 Bridge Geometry

The preliminary bridge geometry and layout criteria are described below.

2.1 Deck Width

A Level of Service (LOS) studies were produced as part of preliminary design.

- Clear deck width between railings shall be 14-ft.

2.2 Alignment Geometrics

- Design speed for the CKC Trail is 15 mph.

- Minimum radius of curvature is set at 50-ft.

- The minimum bicycle stopping sight distance (SSD) shall be 140-ft per AASHTO Bike.

- The maximum cross slope for pedestrian zones is limited to 2% per ADA.

- Maximum grade shall be prescribed as follows per ADA:

    - 5% (1V:20H) without landings (on inside of curve)

2.3 Clearance Requirements

- Vertical clearance over the roadway shall be greater than 16-ft.

- Vertical clearance from the deck surface shall be a minimum of 10'-0".

- Roadway minimum horizontal clear zone to face of unprotected piers is 10-ft.

2.4 Vehicle Stopping Sight Distance

- The bridge design shall ensure vehicular stopping sight distances (SSD) are maintained.

2.5 Railings

- Railing height above the deck surface shall be 4'-6" to accommodate cyclists per the BDM.

- The maximum opening size in the railing is limited to 5 inches.

- Handrails may be provided 34-38" above the walking surface.

2.6 Throw Barrier

- Throw barriers are not required.

2.7 Stairs and Elevators

- Stairs and/or elevators are not currently being considered. If a stairway is desired at the south side of NE 124th St, it could be incorporated into the retained earth wall near the abutment.

2.8 Drainage

- It is anticipated that overpass drainage will be collected at each end of roadways to be piped into roadway storm drains. The impervious bridge deck over the existing impervious roadway will result in a net zero increase in stormwater to the existing roadway drainage system.

- The south approach will drain into vegetation and the north ramp will drain into the parkland of Totem Lake.

2.9 Expansion Joints

- Expansion joints in the travel-way shall be bicycle-safe and detailed to provide a smooth rolling surface with a skid resistant cover plate, installed such that it is recessed and sits flush with the concrete surface. The cover plate will be fixed to the deck on one side of the expansion joint and sit on top of an embedded plate on the other side. The detail will create a single groove gap in the travel way (rather than a raised plate). Steel edges on each side of the gap will be chamfered to minimize the bump felt by wheeled mobility users.
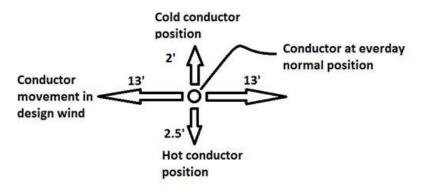
   Cover plates will be galvanized with a non-slip coating applied to the top exposed surface, such as *SlipNot*. Countersunk tamper-resistant screws shall be used to secure the cover plate.

2.10 Utilities and Technology

- Overhead power lines shall maintain sufficient clearance from bridge.

   - PSE Clearances are dependent on what the clearance is being measure to. For human beings, equipment, or cranes, clearances shall meet the WAC minimum approach distances.

- Seattle City Light (SCL) conductor movement envelope for the 230 kV circuit is shown by the following diagram. The clearance required from the conductor in any position to the bridge or bridge elements is 11.5' horizontal and 16.5' vertical. For lighting support, the clearance required is 8.0' horizontal and 8.5' vertical to the conductor in any position. There are currently have no plans for upgrades or replacements to the conductors and these clearances only apply for the area above the CKC because as you move further north, the movement envelope gets bigger.

- Underground utility impacts will be avoided where possible, or rerouted if necessary.

- Four (4) RPVC conduits 2" diameter will be included within the superstructure for future utilities.

- All pathway striping and symbols shall use high quality reflective paint. User delineation is to be determined.

2.11 Lighting

- Low level uniform lighting (5 lux or 0.5 foot-candles, TBC) will be provided along the bridge between at-grade tie-in points with the CKC trail at each end. The lighting system shall be tamper resistant, but readily accessible by maintenance crews.

- A lighting study will be carried out in Phase 2 to demonstrate desired lighting levels are achieved across the bridge, ramps, and along at-grade pathways.

- Glare from the lighting will be minimized over the roadways, with minimal light pollution over the wetlands.

- Accent lighting will be explored in Phase 2 with importance stressed on schemes and colors that do not cause drivers to become distracted or that interfere with the traffic signals at this busy intersection.

3 Design Loads

The applicable loads are based on Table 3.4.1-1 of AASHTO LRFD as modified per AASHTO Ped as follows:

- CR = force effects due to creep

- CT = vehicular collision force

- DC = dead load of structural components and nonstructural attachments

- DD = down drag force

- DW = dead load of wearing surfaces and utilities

- EH = horizontal earth pressure load

- EL = miscellaneous locked in force effects

- ES = earth surcharge load

- EV = vertical pressure from dead load of earth fill

- EQ = earthquake load

- LL = vehicular live load

- LS = live load surcharge

- PL = pedestrian live loading

- SE = force effect due to settlement

- SH = force effects due to shrinkage

- TG = force effect due to temperature gradient

- TU = force effect due to uniform temperature

- WL = wind on live load

- WS = wind on structure

3.1 Vehicular Collision Load (CT)

- Collision loading may be considered in design where no roadside barriers are used along the roadway to offer protection to the bridge piers. The equivalent horizontal static vehicular collision force of 600 kips will be applied five feet

above the ground per LRFD-Spec Section 3.6.5.1. ADT/ADTT for the site will be reviewed to confirm this collision load.

3.2   Dead Loads (DC)

- Prestressed Concrete = 165 pcf per BDM

- CIP Concrete = 155 pcf per BDM (150 pcf per AASHTO LRFD 3.5.1)

- Structural Steel = 490 pcf (per AASHTO LRFD 3.5.1)

3.3   Pedestrian Loads (PL)

- Pedestrian and cyclist live load = 90 psf, placed with patterns to give the maximum load effects.

    - No dynamic load allowance is applied per AASHTO Ped.

    - No reduction in pedestrian loading based on bridge length (this was a previous design consideration in earlier versions of AASHTO).

- The design live load for pedestrian railings shall be taken as w = 0.050 klf, applied simultaneously in the lateral and vertical directions, in accordance with Section 13 of AASHTO LRFD. In addition, each longitudinal element shall be designed for a concentrated load of 0.2 kips, acting simultaneously with the linear load at any point and in any direction at the top of the longitudinal element.

3.4   Vehicle Load (LL)

- Demands for a water tank in tow and/or a mini-street sweeper TBC by the City.

- The bridge deck will be designed to accommodate an H10 two-axle vehicle in accordance with AASHTO Ped Section 3.2. No dynamic load allowance (DLA) will be applied, and the H10 load is exclusive of PL loading. Note that this will allow for maintenance vehicles and small mechanized snow-clearing equipment to be used on the structure.
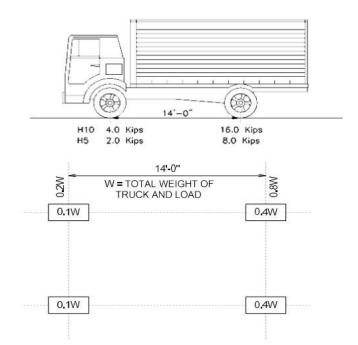
Figure 1: AASHTO Ped Figure 3.2-1 – Maintenance Vehicle Configurations

3.5 Equestrian Load (LL)

- Equestrian load is not considered.

3.6 Wind Load (WS)

- Wind loads shall be per AASHTO LRFD and modified per AASHTO Ped using AASHTO Signs.

- Vertical uplift on bridge deck:

    - 20 psf applied over full deck width acting at the windward quarter point of the deck.

3.7 Earthquake Load (EQ)

In accordance with AASHTO Seismic, the seismic design approach for the bridge will be based on *life safety*. The Response Spectrum will be constructed for the site using LRFD-Spec for the given Site Class as determined by the preliminary geotech recommendations. Potential for soil liquefaction and slope movements will be considered per AASHTO LRFD.

3.8 Temperature (TU, TG)

Uniform temperature loads due to structure temperature fall or rise are defined according to AASHTO LRFD.

3.9 Load Combinations

The bridge will be designed for the load combinations with the load factors as shown in Figure 2 below, which is a modification of the AASHTO LRFD load table in accordance with AASHTO Ped.

| Limit State | Permanent Loads DC DD DW EH EV ES EL PS CR SH | Transient Loads | | | | | | Use one of these at a time | |
		LL PL LS	WS	WL	TU	TG	SE	EQ	CT
STRENGTH IA	γ_p	1.75	-	-	0.5/1.20	-	1.00	-	-
STRENGTH IB	γ_p	1.75	-	-	0.5/1.20	-	1.00	-	-
STRENGTH III	γ_p	-	1.40	-	0.5/1.20	-	1.00	-	-
EXTREME I	γ_p	0.0*	-	-	-	-	-	1.00	-
EXTREME II	γ_p	0.50	-	-	-	-	-	-	1.00
SERVICE I	1.00	1.00	0.30	1.00	1.00/1.20	1.0 (no LL)/0.5 (LL)	1.00	-	-
SERVICE III	1.00	0.80	-	-	1.00/1.20	1.0 (no LL)/0.5 (LL)	1.00	-	-
FATIGUE I - LL, IM & CE ONLY	-	-	1.00	-	-	-	-	-	-

*PL=0 at time of EQ. People on a bridge can be an effective damper when standing still.

Figure 2: Load Combination Table

3.9.1 Strength IA

This combination considers PL and LS loading but not LL.

3.9.2 Strength IB

This combination considers LL and LS without PL.

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3.9.3   Strength III

Wind loading without people present on the deck in such an extreme event.

3.9.4   Extreme I

Earthquake loading of the structure.

3.9.5   Extreme II

Considers vehicle collision on any unprotected bridge piers within the clear zone from the roadway.

3.9.6   Service I

Combination used to check deflections and vibrations per AASHTO Ped.

3.9.7   Service III

Combination used to check tensile stresses in prestressed concrete. This will not be applicable if a steel superstructure is used.

3.9.8   Fatigue I

Fatigue loading shall be applied according to AASHTO Ped Section C3.5, which designates "…wind as a live load for pedestrian bridges, via the designation LL. Wind should be considered a fatigue live load for pedestrian bridges." Section C3.5 further states that, "Neither the pedestrian live load nor the maintenance vehicle load… is appropriate as a fatigue design loading due to the very infrequent nature of this loading."

Therefore, in the table above, WS is assigned a load factor of 1.00 (vs 1.50) per AASHTO Ped, with no fatigue loading arising from LL or PL.

4 Deflection Criteria

In accordance with deflection criteria for pedestrian structures, AASHTO Ped clause 5 will be checked under Service I in Table 3.4.1-1 of AASHTO LRFD. However, AASHTO Ped states in Section 1.1 - Scope that "Pedestrian bridges with cable supports or atypical structural systems are not specifically addressed". It may be the case that provisions associated with the deflection criteria set out in this code are excessively conservative or unachievable for bridges of this type.

5 Vibration Criteria

Vibration analysis is based on AASHTO Ped, which states in Section 6: "If the fundamental frequency cannot satisfy these limitations [of a vertical frequency greater than 3.0 Hz and lateral frequency greater than 1.3 Hz] … an evaluation of the dynamic performance shall be made." The commentary of Section C6 in AASHTO Ped further explains: "The technical guide published by SETRA (Service d'Études Techniques des Routes et Autoroutes) (2006) appears to present a relatively straightforward method for addressing vibration issues when the frequencies of the bridge fall within the pacing frequencies of pedestrians." As AASHTO Ped references SETRA under these conditions, we will perform the vibration analysis using the SETRA method.

5.1 SETRA Determination of Bridge Class

The SETRA vibration analysis method "makes it possible to limit risks of resonance of the structure caused by pedestrian footsteps," as stated in Section 4 of SETRA. The flowchart in Figure 3 gives an overview of the SETRA methodology.
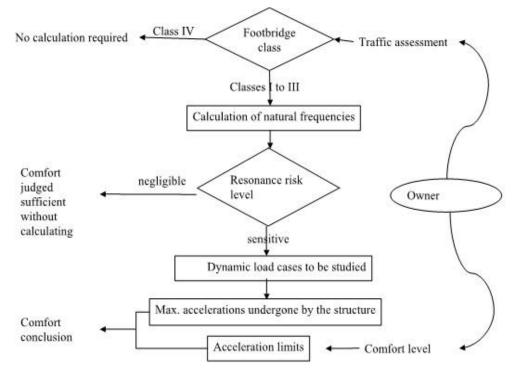

Figure 3: SETRA Methodology organization chart

The method requires that the footbridge class and comfort level be defined, then the natural frequency can be calculated and the appropriate acceleration range can be determined for the bridge. When the calculated natural frequency fits within the assigned acceleration range, the bridge design is considered to meet the vibration criteria.

5.2   Footbridge Class

The methodology first requires a determination of footbridge *Class*, which "makes it possible to determine the level of traffic [the bridge] can bear [comfortably from a vibration point of view]." The footbridge classes are defined as follows:

- **Class IV**: seldom used footbridge, built to link sparsely populated areas or to ensure continuity of the pedestrian footpath in motorway or express lane areas.

- **Class III**: footbridge for standard use, that may occasionally be crossed by large groups of people but that will never be loaded throughout its bearing area.

- **Class II**: urban footbridge linking up populated areas, subjected to heavy traffic and that may occasionally be loaded throughout its bearing area.

- **Class I**: urban footbridge linking up high pedestrian density areas or that is frequently used by dense crowds subjected to very heavy traffic.

The Totem Lake Connector Bridge is designated as Class III based on projected use levels of the CKC trail. Note that it is important to keep in mind that these definitions are for *comfort* and have no effect on the loading used in structural design. Structural design considers the bridge being fully loaded throughout its applicable bearing area per Section 3.3.

5.3   Comfort Level

The level of comfort also needs to be determined according to the following SETRA definitions:

- **Maximum comfort**: accelerations undergone by the structure are practically imperceptible to the users.

- **Average comfort**: accelerations undergone by the structure are merely perceptible to the users.

- **Minimum comfort**: under loading configurations that seldom occur, accelerations undergone by the structure are perceived by the users, but do not become intolerable.

The Maximum comfort designation is most appropriate for pedestrian bridges linking transit centers or a stadium.

There is also a psychological aspect associated with comfort. Users of a cable supported structure (e.g., suspension bridge, cable-stayed) tend to expect more movement compared to crossing a girder or truss bridge. The definition of comfort is therefore somewhat variable with respect to structure type. In any event, for this type of urban bridge all concepts should meet the Minimum comfort level.

Thus, we propose the bridge be designed to the Average or Minimum comfort level based on the concept type.

6 Foundation Considerations

6.1 Abutments

Bridge abutments will form the interface between aerial structure and pathways on fill.

6.2 Approach Slabs

Approach slabs approximately 10-ft long will be provided at the south and north abutments to ensure smooth transition on and off the bridge.

6.3 South Approach

For the south approach, use of terraced vertical or battered earth walls may be explored as a cost effective solution. Fill embankments and/or vegetated reinforced earth slopes of 2H:1V or flatter could also be employed. Lightweight fill could be used to limit settlements, or the main bridge could be extended to reduce fill height if poor subsurface conditions are discovered.

6.4 Foundation Types

The main superstructure support will be provided on deep foundations (piles or drilled shafts), with shallow spread footings possible south of Totem Lake Blvd.

6.5 Drilled Shafts

Drilled shaft design will be as follows:

- Single drilled shafts will be used where practical.

- Drilled shafts will be designed for controlling of English vs. Metric diameter sizes. Metric casing sizes will be based on WSDOT BDM Table 7.8.2-2. Plans will detail shafts for English size, and contractor will be allowed to substitute equivalent metric sizes.

- Drilled shaft lengths shall be set to meet the following:

    - Provide sufficient geotechnical axial capacity.

    - Provide sufficient depth for lateral loads as follows:

        - Per FHWA Drilled Shafts, lateral deflections at the top of the shaft shall be limited to 10% of the shaft diameter.

- For a given loading, the minimum shaft depth will be set so that the top of shaft deflection is not sensitive to or significantly influenced by the depth of the shaft. The shaft depth vs. deflection curve shall be examined to ensure the shaft depth is at or below the point at which the curve begins to flatten; i.e., the depth at which the top defection is significantly influenced by the depth of the shaft.

  - A tip of shaft deflection less that ½-inch will be targeted for the design seismic event.

- Drilled shaft concrete cover and clearances will be based on WSDOT BDM Table 7.8.2-2.

- Rebar spacing limits will be in accordance with WSDOT BDM requirements.

- Drilled shaft demand will be controlled by plastic hinging in the columns. Flexural and shear capacity will be checked based on the following:

  - For flexural and shear capacity calculations, concrete strength = 0.85 f'c

  - Flexural capacity check as follows:

    - $M_{ne} > 1.25\ M_{po}$

    - SPColumn shall be used to determine M_{ne} using expected properties per AASHTO Seismic Table 8.4.2-1 as modified by WSDOT BDM 4.2.22 if using ASTM A706 Gr. 80 reinforcing.

    - Flexural capacity will be based on expected nominal capacity, M_{ne}, and determined based on a moment curvature analysis.

    - M_{po} = maximum moment demand based on column plastic hinging with overstrength factor.

    - Additional 1.25 factor applied to M_{po} is per AASHTO Seismic Section 8.9.

  - Shear capacity check as follows:

    - $\Phi_s V_n > 1.25\ V_{po}$

    - $\Phi_s = 0.90$

    - V_{po} = maximum shear demand based on column plastic hinging with overstrength factor.

    - Additional 1.25 factor applied to V_{po} per AASHTO Seismic Section 8.9.

7 Analysis

The following types of analysis will be used for design. Model descriptions are provided in the following sections. Software (indicated in parentheses) will supplement hand calculations and spreadsheets.

(1) Global model(s): for general purpose analysis to model DL, PL, LL, Wind, EQ, etc. (CSI Bridge and/or CAMIL)

(2) Local models: for detailed design of particular elements (CSI Bridge)

(3) Foundation models: for determining foundation springs used in the global model(s) (L-PILE)

(4) Pushover analysis: to determine column displacement capacities and plastic hinging loads (CSI Bridge/XTRACT)

(5) Substructure: biaxial capacity (SP Column)

7.1 Global Models

Global modelling will be as follows:

- CSI Bridge or CAMIL (COWI's in-house software with specialized capabilities for cable structures) will be used to model the structure.

- The bottom of columns will be modeled with an equivalent soil spring or the foundation will be modelled with P-y soil curves through depth.

- Effective section properties:

  - All elements except columns will use gross moment of inertia for all loading.

  - Concrete piers:

    - For vibrational analysis, the gross moment of inertia will be used, as displacements are small.

    - For strength analysis, the effective (cracked) moment of inertia will be used.

    - Effective moment of inertia for temperature load cases = $0.6 \cdot I_g$.

    - Effective moment of inertia for seismic design = $0.4 \cdot I_g$.

- Once reinforcing in columns is designed, effective moments of inertia will be verified with a strain analysis per BDM page 4-B2-23, Appendix 4-B2 Section 3.1.2 Cracking of Columns.

- Use the effective torsional moment of inertia for all load cases based on AASHTO Seismic Section 5.6.5 for estimating $J_{eff} = 0.2 \cdot J_g$.

- Foundation Springs

  - The global model will extend down to bottom of columns or top of foundation. At these locations, foundation springs may be used to model the equivalent foundation stiffness.

  - See section 7.3 Foundation Models for details on developing foundation springs.

- Response Spectrum Analysis

  - Elastic analysis with cracked section properties.

7.2   Local Models

Local models will be used to study elements such as abutments, cable connections, piers, etc.

7.3   Foundations Models

Foundations will be modeled by either directly including the shaft or pile with associated P-y curves or through a lumped equivalent spring representing the foundation stiffness. This is done as follows:

- Foundation springs will vary depending on displacement amplitude and soil properties. Soil properties used for developing springs at each column location will be based on recommendations provided by the project Geotechnical engineer.

- Results between L-PILE and CSI Bridge will be iterated until the displacements and loads converge. Since soil response along the length of shafts is non-linear, iteration is required with the results from the global model. Results between the global and foundation models are concluded to have converged when they are within 10%.

- Stiff vs. soft foundation analysis: sometimes two separate seismic analyses are required to bound the foundation response. Generally, a stiff foundation

response will provide the largest column demands and control column design. A soft foundation will control displacements. During the design, consideration will be given to which type of analysis is appropriate. Shaft properties considering stiff vs. soft foundations will be calculated using BDM Section 7.2.3.

- For a given foundation, the same springs should be used for as many loads as reasonable to avoid excessively complex analysis and iterations.

7.4   Pushover Analysis

To verify seismic displacement capacities of the bridge substructure, a non-linear pushover analysis will be performed on a representative pier.

The purpose of pushover analysis is to determine column displacement capacities and plastic hinging loads for shaft design.

- Software: CSI Bridge/XTRACT.
- Soil springs should be included in pushover analysis.

7.5   Substructure Analysis

This analysis may be used to assess vehicle collision loads on unprotected piers.