PB2002-106733

RAPID REPAIR DESIGN OF TEMPORARY SUPPORT SYSTEMS FOR BRIDGES DAMAGED BY EARTHQUAKES IN THE STATE OF WASHINGTON

WA-RD 542.1

Final Report
October 2001

Washington State Department of Transportation

Washington State Transportation Commission
Planning and Capital Program Management
in cooperation with:
U.S. DOT - Federal Highway Administration

Final Report

Research Project Contract No. T1804, Task 9
Emergency Bridge Repair

RAPID REPAIR DESIGN OF TEMPORARY SUPPORT SYSTEMS FOR BRIDGES DAMAGED BY EARTHQUAKES IN THE STATE OF WASHINGTON

By

Rafik Y. Itani
Professor

Kenneth J. Fridley
Professor

Luke A. Heath
Graduate Research Assistant

Washington State Transportation Center (TRAC)
Washington State University
Department of Civil and Environmental Engineering
Pullman, WA 99164-2910

Washington State Department of Transportation
Technical Monitor
Mark Szewcik
Bridge Engineer

Prepared for

Washington State Transportation Commission
Department of Transportation
U.S. Department of Transportation
Federal Highway Administration

October 2001

TECHINICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.

4. TITLE AND SUBTITLE	5. REPORT DATE
Rapid Design of Temporary Support Systems for Bridges Damaged by Earthquakes in the State of Washington	October 2001
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Rafik Y. Itani and Luke A. Heath	8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Washington State Transportation Center (TRAC) Civil and Environmental Engineering Sloan Hall, Room 101 Washington State University Pullman, Washington 99164-2910 Mark Szewcik, Technical Monitor 360-705-6814	10. WORK UNIT NO.
	11. CONTRACT OR GRANT NO. T1804-9
12. SPONSORING AGENCY NAME AND ADDRESS Research Office Washington State Department of Transportation Transportation Building, MS 7370 Olympia, Washington 98504-7307	13. TYPE OF REPORT AND PERIOD COVERED Final Report
	14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation and Federal High Administration.

16. ABSTRACT

The scope of this project is to provide designs for the rapid construction of shoring systems for damaged bridges due to earthquakes in the state of Washington. Using a broad range of loading criteria and established bridge geometries, several shoring systems were developed for 75% of the bridge types in Washington that are encountered in typical field scenarios. Using materials commonly available in the Puget Sound stockyards, steel and a wood/steel combination shoring system were developed to shore standardized concrete pre-stressed girder bridges, concrete box girder bridges, or steel plate girder bridges. The temporary shoring can span a height of 15 to 40-feet. A handbook, with a flowchart and detailed construction drawings, was developed to provide WSDOT inspection engineers with tools to administer the full, appropriate completion of the shoring systems to allow the public and emergency crews to traverse damaged bridges.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Shoring systems, bridges, earthquake, pre-engineered plans	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None		

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Transportation Commission, Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Summary

The scope of this project is to provide designs for the rapid construction of shoring systems for damaged bridges due to earthquakes. Currently, there are no pre-engineered, standardized shoring plans and the majority of all decisions for the disposition of earthquake-damaged bridge structures are made within the first ten days of a declared national emergency. Standard practices throughout the world involve designing temporary shoring systems for individual bridge cases, which ultimately taxes available time, resources, and safety of the public.

The Washington State Department of Transportation has determined that restoring bridges to a serviceable level as quickly as possible is in the best interest of emergency crews, the public, and the State economy. 75% of all bridges within the state of Washington are standardized concrete pre-stressed girder bridges, concrete box girder bridges, or steel plate girder bridges. As a result, a general set of pre-engineered shoring plans could be made for each of these three types of standardized bridges. Consequently, less time, money, and resources will be spent on design and construction of a shoring system and engineers could focus on permanent solutions to repairing bridges. Also, the public and emergency crews will have a traversable bridge to use in the event the bridge is damaged at the columns, since the shoring system will act as a column.

Using a broad range of loading criteria and established bridge geometries, several shoring systems were developed for the array of bridge types and cases that are encountered in typical field scenarios. Using materials commonly available in the Puget Sound stockyards, steel and a wood/steel combination shoring system were developed to shore the aforementioned bridge types. The temporary shoring can span a height of 15 to 40-feet.

A handbook, with a flowchart and detailed construction drawings, was developed to provide WSDOT inspection engineers with tools to administer the full, appropriate completion of the shoring systems.

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Design Manual for the Rapid Construction of Temporary Shoring Systems A1

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Purpose and Scope

This project provides a design manual that includes guidelines and procedures for constructing rapid temporary shoring systems for damaged bridges. This manual will be further integrated into the WSDOT Bridge Design Manual. The designs provided here will be used by the WSDOT bridge officials who will oversee all construction of the temporary shoring systems. This manual could also be implemented into the FEMA handouts for bridge inspection and could be provided to assist other local governments.

WSDOT responsibilities after an earthquake that damages the state transportation system are to insure the safety of the traveling public, protect transportation facilities from further damage, restore traffic on the highway system as quickly and safely as possible, and to maintain a timely and current assessment of the extent of damage and operation status of the transportation system.

The scope of this project deals primarily with the technical aspects of building a rapid pre-engineered shoring system for damaged bridges. Broad criteria were set forth by the WSDOT Bridge and Structures Office to accommodate 75% of all bridges within the state of Washington. The shoring system is to retain heights between 15 to 40-feet.

Background

Preparedness to respond to natural disasters and restoring normalcy to living conditions are essential for recovering socially and economically from such hazards. Communities through out the world have plans to quickly repair damage to lifelines such as water supply system, power system and transportation facilities. One essential plan must deal with restoring transportation systems particularly when damages to bridges will have significant influence on the continued safety of the community and the interruption

to continued economic activities. A plan for the construction of temporary support systems of damaged bridges is needed. Such a plan will include the quick and easy response to supporting damaged bridges through safe temporary supports that are easily erected in the field and which will utilize nearby resources and materials.

Benefits

The immediate benefits of this project are the ability to restore normalcy and economic lifelines of an affected community. This project will provide the necessary rapid design methods and detailed construction plans to quickly restore traffic across damaged bridges. These designs will be shared with towns and cities throughout the State of Washington as well as other states pursuing the development of similar processes to deal with natural hazards. Coordination of the development of such designs and construction details will also be conducted in cooperation with the construction and engineering community.

Design Criteria

The Seattle corridor along the north-south freeways if dilapidated by an earthquake, could devastate the economy of Washington. Since most earthquakes in the state occur on the West Coast at the subduction zone of the Juan de Fuca fault, materials, material availability, bridge types, and loading conditions were considered in determining the appropriate design criteria that are common to Western Washington. Under these considerations, the following design criteria was established and developed with WSDOT bridge engineers to address conditions relevant to the western parts of the state:

- Loading will be HS-20-44 using AASHTO provisions
- Design must address at least 75% of state bridges in western Washington

- Hem-Fir or Douglas Fir-Larch will be used for vertical supports and standard steel beams for horizontal members. Design for standard beams will also be provided for vertical supports. Tables with steel post options, as well as for wood post options will be prepared
- Prestressed concrete girder and concrete box girder bridges will be addressed in the study. Also steel plate girder bridges will be analyzed.
- Lateral bracing to support 2% of the dead load of the bridge applied horizontally.
- Load combinations for Allowable Stress Design shall include Dead + Live load for posts and mudsills, and Dead + Live + Impact load for horizontal members directly supporting the superstructure. For Load and Resistance and Factor Design, the load combinations shall be 1.2D+1.6L for posts and mudsills, and 1.2D+1.6L+1.25I for horizontal members directly supporting the superstructure.
- Height of temporary shoring shall be from 15' to 40 feet
- A base value of super-elevation will be considered. Any roadway super elevation exceeding the base value will require an adjustment in design
- Use the English system of units
- Assemble material inventories available for shoring located within the construction yards of Puget Sound contractors.
- All Dead, Live and Impact loads are to be multiplied by a factor of safety of 1.25 for Allowable Stress Design (ASD) and for Load Resistance and Factor Design (LRFD).

Design Values

Listed below is a summary of the material allowable stresses and the density of the pertinent materials used in load calculations.

Wood Values

$F_c = 1000$ lb/in^2

$F_b = 1100$ lb/in^2

$F_{cperp} = 400$ lb/in^2

$E = 1{,}300{,}000$ lb/in^2

$F_t = 600$ lb/in^2

$F_v = 110$ lb/in^2

Soil Values

Bearing Pressure = 3000 lb/ft^2

Steel Values

$F_y = 36$ kips/in^2

$E = 29{,}000$ kips/in^2

Density = 490 lb/ft^3

Concrete Values

$F'_c = 4000$ lb/in^2

Density = 160 lb/ft^3

All steel is A36 steel. The following is a summary of the materials used to construct the different shoring towers.

Materials Used for Wood Column Shoring Tower

Columns = 12x12 Hem-Fir/Douglas Fir-Larch or 12" minimum diameter timber pole

Cross Bracing = 4x8 Hem-Fir/Douglas Fir-Larch or larger

Bolts = ¾ " and 1" diameter A307 bolts

Welds = ¼" Fillet with E70 Electrodes

Transverse Cap = HP12X53 or HP14X89

Sill Beam = HP12X53 or HP14X89

Double Plate Stiffeners for Steel Girder Bridge = 6"x72"x½"

Steel Shims or Wedges

Materials Used for Steel Column Shoring Tower

Columns = HP12X53 or HP14X89

Cross Bracing = Angle 4X4X½

= Channel C3X6

Base Plates = 1" thick A36 Steel

Welds = ¼" Fillet with E70 Electrodes

Transverse Cap = HP14X89 (Box Girder Bridge only)

Sill Beam = HP14X89

Corbels = HP12X53 or HP14X89

Foundation Plates = 5'x10'x1" A36 Steel

= 12x12 Hem-Fir/Douglas Fir-Larch

Double Plate Stiffeners for Steel Girder Bridge = 6x6x½ Angles

Steel Shims or Wedges

Assumptions

The following assumptions were made pertaining to the temporary shoring system and its conditions:

- It is assumed that there are to be no splices for the wood shoring columns. Typically, 12x12 Hem-fir comes in lengths up to twenty feet while timber poles come in heights up to 40-feet or more. Timber poles can be used for 15 to 40-feet. The timber poles must meet dimension requirements that are stated within ASTM Standard D3200.
- All columns are to be centered and placed plumb as depicted in the member layout drawings. Steel shims and jacking struts shall be used to make the columns plumb.
- All wood members are to be Hem-Fir or Douglas Fir-Larch and No. 2 or better.
- Lateral bracing is designed to resist lateral loads equivalent to 2% of the dead loads. The lateral bracing will also help by reducing the column's unbraced length.
- The members listed or specified in the drawing layouts or tables, may be replaced by higher capacity members. For example, the cross bracing in the wood column

shoring tower calls for a 4x8 Hem-Fir. A higher grade of Hem-Fir or Douglas Fir-Larch of the same size or a larger member like a 4x12 with the same grade of wood may be substituted for the required cross bracing member while the layout of the members remains the same. Any change in the layout of members may not be made without the prior approval of a licensed engineer.

- Diagonal cross bracing for the wood column structures may be at angles of 20 to 45-degrees measured with respect to the horizontal plane. For the steel columns, the cross bracing is assumed to be at 45-degrees.
- Tower structures are to be placed as close to the existing damaged columns as deemed safe and feasible by the engineer of record.
- Tremors or other small earthquakes followed by the large catastrophic earthquake are not part of the loading criteria of the shoring system.
- The weight of the shoring system is considered small in comparison to the applied loads and is ignored in the design of the footings.
- There is to be no field welding to the existing steel plate girder bridge.

Bridge Descriptions

Seventy-five percent of all bridges in Washington are standard reinforced concrete box girder bridges, prestressed concrete girder bridges or steel plate girder bridges. By selecting the critical bridge in each category, the shoring system could be developed to accommodate all bridges of the corresponding subset. For that reason, an existing, critical bridge from each category was chosen to be modeled for the shoring systems.

For each of the three bridges, the dead, the live, and the impact loads were determined. The dead load was calculated based upon the geometry and material properties. For reinforced concrete and steel, the dead load was computed using a density of 160 lb/foot^3 and 490 lb/foot^3, respectively. The live load was calculated using the AASHTO (American Association of State Highway and Transportation Officials) Standard Specification for Highway Bridges procedures by applying HS20-44 loading using either lane or truck loading, according to section 3.11-12. The lane or truck loading was employed depending on which ever created a critical load for the shoring system. The AASHTO provisions were also applied to determine the impact load using the impact formula set forth in section 3.8.2.1. The designed shoring system is applicable to the same type of bridge of equivalence or smaller size. The following is a description of the modeled bridges:

Reinforced Concrete Box Girder Bridge

The reinforced concrete box girder is located on State Route 18 in King County. The modeled bridge has two spans of 110-feet and a center span of 140-feet. The box girder is 7-feet in depth and has a width of 54-feet and 2-inches on the underside of the superstructure. There are 6 box girder bays that comprise the width of the bridge. This particular bridge has a four-lane capacity. The deck has a thickness of 6.5-inches.

Prestressed Concrete Girder Bridge

The prestressed concrete girder bridge is located over Tye River on State Route 2 in King County. This bridge is simply supported and is 140-feet in length. There are eight W74G standard girders that make a deck width of 48-feet to provide three lanes of travel. The deck has a thickness of 7.5-inches.

Steel Plate Girder Bridge

The multi-span steel plate girder bridge is located in Seattle on Steel Hill Road. The spans are, correspondingly, 67.5, 155, 79, 107, 133.5, and 107.5-feet long. The critical dead load section was analyzed on the 155-foot span. This particular span included a hinge on the two-lane bridge. The hinge was ignored both in the analysis and for the dead load calculations. There are three 72"x5/16" steel plate girders that support the concrete deck and adjacent sidewalks. The concrete deck has a thickness of 6.75-inches with a bridge deck width of 26 feet. When determining the dead load of the bridge, the nuts, bolts, welds, and shear studs were ignored. The maximum live load for the shoring systems was established when the bridge acted just as a two span bridge with spans of 133.5 feet and 107.5 feet, instead of a 6 span bridge.

The following table provides a summary of the calculated loads at the critical reaction for the temporary shoring system using the previously discussed methods and without applying the 1.25 factor of safety.

Calculated Loads without any Factor of Safety per reaction in kips			
Type of Bridge	**Dead Load**	**Live Load**	**Impact Load**
Box Girder	3070.59	345.81	65.44
Prestressed Girder	1100.80	191.16	36.06
Steel Plate Girder	891.92	233.68	40.58

All-Wood Shoring Tower

An all-wood shoring tower was considered to help support the damaged superstructure. This shoring would be a benefit due to an abundant supply of timber in the Pacific Northwest. However, the high loads created difficulty for the timber members, especially for the bending members. Wood is also very tenuous when loaded

perpendicular to grain. As a result of these two hindering factors, large quantities of wood would have to be used to accommodate the high demands of the dead and live load. Thus, a modified wood column shoring tower was developed with steel components to supplement the material strength properties of wood.

Wood Column Shoring Tower

To maximize the use of materials, a wood column shoring tower was incorporated with steel bending members. Design of wood members for the wood column shoring system is based upon the 1997 NDS (National Design Specification), which uses an allowable stress design. Since the shoring system could be in place for up to one year, a load duration factor of 1.1 was applied. The load duration factor for cross bracing members was conservatively taken to be 1.33 instead of 1.6 for wind. The steel members in the wood column shoring system are contrived by the allowable stress design methodology set forth in 1989 9th Edition of the AISC (American Institute of Steel Construction, Inc.) Manual.

The wood column-shoring tower was designed (Sheet No. 1) using 117 kip load per column as determined from the critical load combination of the steel plate girder bridge. Two types of columns are acceptable for the tower, a 12x12 Hem Fir or Douglas Fir-Larch or a 12-inch minimum round. However, there are height limitations set upon the 12x12 columns, since none of the columns may be spliced. Typically, 12x12 come in lengths up to 20 feet. For shoring heights larger than 20 feet, timber poles will be used since they can come up in lengths to the required 40 feet. At 5-foot and 10-foot column unbraced lengths, the 12x12 wood column has a capacity of 143 kips and 134 kips, respectively. At a 5-foot column unbraced length, the 12-inch minimum round wood

column has a capacity of 138 kips and 158 kips for a height of 15 feet and 40 feet respectively. There is large reserve strength left within either types of shoring columns to withstand dynamic loads of vehicles across the bridge. Both column types were chosen to have 5-foot bracing length along the member to create a stiffer shoring tower and provide a larger factor of safety.

The shoring system, as required, must resist lateral forces equivalent to 2% of the dead load. A more conservative approach was applied using the dead plus live loads to resist lateral forces. The cross bracing was designed to withhold tension stresses in the member. Compression stresses were also considered in the design. The cross bracing members are 4x8 and were acceptable for both tension and compression. A 10-foot Hem-Fir or Douglas Fir-Larch with a 4x8 cross bracing member can hold 6% and 8% of the dead load in compression and tension respectively at a maximum angle of 45-degree. The cross bracing capacity is, however, governed by the bolted connection.

The bolted connections were designed using the Hankinson Equation and the European Yield Model stated explicitly in the NDS. To accommodate a large range of construction layout scenarios, the diagonal cross bracing members (Sheet No. 6, Section 7) are to be placed at angles between 15 and 45-degrees. The diagonal cross bracing connection with two A307 7/8-inch or two ASTM A307 1-inch diameter bolts has a capacity to hold 3% and 3.5% of the dead load, respectively. This meets the requirement to resist the equivalent lateral load of 2% of the dead load. One-inch diameter bolts are recommended for the design of diagonal cross bracing members.

The horizontal cross bracing members (Sheet No. 6, Section 7) were designed to provide stability and stiffness for the structure. The horizontal cross bracing member

uses two ASTM A307 ¾-inch diameter bolts with a 4x8 Hem-Fir or Douglas Fir-Larch member. The bolted connection has a capacity to withstand 2.5% of the dead load, which is more than the required 2% as stated in the design criteria. The horizontal members were also designed to resist tension.

The transverse cap (Sheet No. 2, 3, & 4) and sill beam (Sheet No. 5) are made up of either HP12X53 or HP14X89 with A36 steel. Applying the design equations stated within the ASD steel manual, the HP12X53 was found to have a shear and moment capacity of 73.8 kips and 125.1 kip-feet respectively. The HP14X89 has a shear and moment capacity of 122.5 kips and 207.7 kip-feet respectively. Both steel members are sufficient for the different loading cases and member layouts. All steel members are permitted to be continuous under the criterion that there is enough steel to meet the provisions set forth in the graphical depictions of the shoring layout. The H-Piles are not required to have any bearing stiffeners for the purposes of web crippling, local web yielding, compression of the web, or for shear deficiencies for the defined loads at all points of concentrated bearing.

For the box girder, a uniform load is applied to the transverse cap (Sheet No. 3). For either a continuous or segmented beam, the maximum required shear and moment are 63 kips and 52.5 foot-kips, respectively. This is less than the nominal strength of the HP12X53 and HP14X89. Since the box girder has less dead load and live load than the critical steel girder, a less severe dead plus live plus impact loads of 115 kips was applied in place of the critical load of 125 kips from the steel plate girder bridge. The concrete prestressed girder and the steel plate girder bridge will have concentrated loads at the

transverse cap, but as mentioned, no bearing stiffeners are required for the dead plus live plus impact loads of 125 kips.

The shoring foundation (Sheet No. 5) uses 12x12 timbers for the footings and a steel sill beam to distribute the load from each column. The shear capacity, which governs the design, has an allowable and required stress of 110 psi in the wood foundations. The timber foundation member has a bending stress capacity of 1210 psi allowable stress and is only subjected to 980 psi for the critical loading case. The steel sill beam can be either a HP12X53 or HP14X89. For either sill beam, no bearing stiffeners are required for the given loads. The bearing area is 46 square feet with the 6-12x12 that are 8-feet long. The 12x12 wood members may be continuous with one other column, assuming that the footings meet the length requirement. For example, if the footing requires an 8-foot span for one column, the columns must be spaced at 8 feet apart and the 12x12 footing member must be 16 feet long for the continuous footing. The soil is assumed to have an allowable bearing pressure of 3000 pounds per square foot. Each footing is designed to hold 117 kips, which neglects the self weight of the shoring structure. The sill beam met the shear and moment demands of 48.8 kips and 60.9 kip-feet.

In order to insure proper construction of the shoring system, steel jacking struts will be used to elevate the damaged bridge at the site of where the shoring system is to be placed. Generally, an equivalent load of 10% of the dead load is applied to the jacking struts. One hundred and fifty or 250 ton bottle jacks are used with the steel jacking struts. After the pre-load has been applied using the bottle jacks, steel shims or steel wedges shall be placed underneath the columns to ensure that the columns are plumb and abutted

securely to the bridge and the foundation. Shoring columns closest to the damaged columns or support will be preloaded more to distribute load equally with the outer most shoring columns. The cross bracing should then be rechecked to ensure that the connections are tight and have not loosened during this procedure.

Each shoring column end connection (Sheet No. 6) to the transverse cap or sill beam is to be thoroughly secured so that the columns will not displace during their temporary service. The welded angles and through bolts or welded steel cans are sufficient to prevent the columns from lateral displacement. There are a variety of end connections that will limit displacement of column ends. Any approved method is acceptable.

Full-height stiffeners (Sheet No. 6, Section 3) are utilized to ensure that the steel plate girders do not buckle under the concentrated load. The angle stiffeners are to be placed concentrically at the point of contact between the column and the steel plate girder, one on each side of the web of the steel section. The stiffeners are to be 6x6x½ angles and full web height. The angles are to be bolted with 7/8" diameter A325 bolts and spaced on center at 4-inches. The angles have a 1-inch cope.

The critical axial load for one column in the shoring system was 117 kips, which was determined from the steel plate girder bridge. The factored loads employed to calculate stresses in the members are as follows:

ASD Loads with 1.25 Factor of Safety per side of reaction in kips			
Type of Bridge	**Dead Load**	**Live Load**	**Impact Load**
Box Girder	1919.12	216.13	40.90
Prestressed Girder	1376.00	238.95	45.08
Steel Plate Girder	557.45	146.05	26.10

Steel Column Shoring Tower

A steel shoring tower was also considered, since most localities in Western Washington have access to structural steel. The most common structural steel for construction purposes is the H-Piles. The steel column shoring tower (Sheet No. 7) uses two types of H-Piles, HP12X53 and/or HP14X89. Each column is designed to hold 269 kips. HP12X53 and HP14X89 columns have an axial capacity of 420.7 and 731.6 kips, respectively for a 20-foot and 10-foot bracing member about its strong and weak axes. Both members are sufficient to carry the axial loads generated by the various load cases. The steel column shoring tower was designed by the load and resistance and factor design methodology stated in the 1998 edition of the AISC Manual for steel construction.

The foundations (Sheet No. 10) were designed under several assumptions. First, the 269 kip point load is assumed to distribute the load evenly among the corresponding footing members. Secondly, the soil is assumed to resist the load in a uniform manner, with a bearing capacity of 3000 lb/ft^2. In order for the soil pressure to act uniformly, the steel plates had to have a minimal deflection. The 5'x10'x1" steel plates distribute the axial load to the ground. The maximum displacement is 0.12-inch for the steel plate, which is acceptable to assume uniform distribution of stresses. The 12x12 Hem-firs distribute the soil pressures to the steel plate uniformly. The foundation design also utilizes HP14X89 for the sill and corbel beams. The 1-inch thick plates that the columns rest upon are designed to only hold the allotted load of 269 kips.

As required, the cross bracing members (Sheet No. 11) must resist lateral forces of 2% of the axial dead load. However, the members were designed to resist 5% of the 300 kips in tension, according to the provisions set forth in AISC Section J. The 4x4x½

steel angles hold 25% of the design load, while the welded connection resists only the required 5%. The C3X6 channel has a tension and a weld capacity to resist 33% and 15% of the tensile load, respectively. A more efficient design for the channel could be chosen, but this is the smallest practical available size. The cross bracing members are adequate for this shoring system.

A transverse cap (Sheet No. 8) is needed to stabilize the shoring tower from lateral displacements and help transfer the design load to the column members. The HP14X89 was chosen since the HP12X53 does not have the required moment and shear capacity to resist the design loads. The punching shear was checked for the bottom of the concrete box girder using a concrete strength f'_c of 4000 lb/in^2. The box girder had a capacity of 439 kips, which is less than the required 536 kips. While the needed strength of the concrete slab is less than required, there are 2 box girder walls that will prevent punching shear failure. The concrete diaphragm walls will distribute most of the load through the webs so that the bottom slab will not fail by crushing. Also, if the as-built dimensions for the damaged box girder are known, the shoring plans explicitly state that the shoring columns will be placed directly under the webs. For these reasons, the transverse beam is acceptable.

Full-length stiffeners (Sheet No. 11, Section 3) are required to ensure that the steel plate girders on the bridge do not buckle under the concentrated load. They are designed for the critical load combination total of 151 kips. The stiffeners are 6x6x½ angles and span the full height of the web. They have a 1-inch cope to allow for the fillet welds between the web and the flange. One pair of stiffeners can hold 254 kips, which is sufficient for the steel plate girders.

As a result of the geometry and loads of the concrete prestressed girder bridges, the wood column shoring system was chosen rather than the steel column shoring system. The wood column shoring system is more efficient than the steel columns since its capacity matches the required loads supporting the bridge superstructure. Should the steel tower be installed, the transverse beams would have to span across several girders and resist large values of shear and moment. The HP12X53 and the HP14X89 are incapable of supporting the required bending loads. For that reason, the steel column shoring system is not recommended and no further calculations are provided.

The steel plate girder bridge (Sheet No. 9) was designed with only one column per girder initially. However, due to the lateral force criterion on the structure, a frame was designed rather than a single row of columns. In fact, the columns did not need lateral bracing to resist axial buckling. As a result, the shoring system for the steel girder is inefficient, but acceptable.

In general, the steel shoring tower is not an efficient use of materials, but it is an acceptable shoring system to employ for the specified bridges. To make construction efficient, ¼-inch fillet E70 welding is applied to all connections. As discussed in the Wood Column Shoring Tower section, a jacking strut system must be incorporated to facilitate proper construction of the shoring system. The factored loads for the steel column shoring tower are as follows:

Controlling LRFD Loads per side of reaction in kips			
Type of Bridge	**1.4*DL**	**Live Load 1.2*DL+1.6*LL**	**Impact Load****
Box Girder	2686.6	Not controlling	40.90
Steel Plate Girder	Not controlling	902.8	26.10

**Impact Load is multiplied by 1.25 DL = Dead Load and LL = Live Load

Suggestions and Conclusions

There are a couple of suggestions that should be considered during and after the construction of the temporary shoring systems. Vehicular speeds should be reduced to 15 mph to minimize the dynamic effects on the bridge superstructure and the temporary shoring system. However, the speed reduction may not be practical in high volumetric demand areas. Therefore, there should be constant monitoring of cracks, settlement, and connections on the super-structure and also on the temporary shoring system for safety purposes.

The shoring towers have large safety factors applied to the acquired loads. Undoubtedly, the shoring systems are very conservative designs. The factor of safety will be of benefit since there are many uncertainties that cannot be accounted for such as post tremors, construction equipment loads, dynamic loading, soil capacity, etc.

In conclusion, the wood column shoring tower will be of more benefit to WSDOT for several reasons. First, there is an abundant supply of wood columns due to the many forests in the Pacific Northwest. Secondly, the utilization of wood columns is more efficient because of a closer match between resistances and capacities of structural members. Finally, a wood column shoring tower can be easily constructed and requires less skillful labor than other materials such as those associated with welding. Thus, the wood column shoring tower will be used more often than the steel column shoring tower.

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