TASK SUBMISSION

TASK 6 - GUIDE SPECIFICATION AND RETROFIT MANUAL
TASK 6b – 90%RETROFIT MANUAL

September 7, 2007

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS AND
HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

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HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

1. PURPOSE

1.0 Introduction

The purpose of this Handbook is to provide:

- a method for screening a bridge inventory to identify those structures potentially vulnerable to coastal storm events
- a method for evaluating the identified structures to determine the specific vulnerabilities
- potential retrofit strategies, approaches, and measures to address any uncovered vulnerabilities.

1.1 Screening

The objective of the screening procedure is to provide a reasonably simple method for determining which bridges in an inventory are most potentially vulnerable to coastal storm events. The methodology used in this manual consists of calculating a simplified vertical force estimate due to wave and surge effects, and comparing to the available vertical resistance of a structure. This vulnerability index is then combined with an importance index resulting in a prioritization ranking.

1.2 Evaluation

The evaluation of an existing structure is similar to the evaluation step in the design process for a new coastal structure as specified in the Guide Specification for Bridges Vulnerable to Coastal Storms (Guide Specification). Adjustments are made to the evaluation process to account for the specific needs of retrofit analysis.

1.3 Retrofit Strategies, Approaches, and Measures

It is the intent of this section to present retrofit concepts that may be used to make vulnerable bridges more resistant to coastal storms. Many of the factors affecting the choice of a retrofit solution will be unique to each bridge. A wide variety of retrofit options are detailed in this section; at least one method should prove to be viable for each individual structure. Variations and combinations of the retrofit options shown, along with other innovative ideas, are likely to provide the most suitable solution.

Development of cost-effective retrofit options for an existing bridge is an iterative process as shown in the flow chart of figure 1.3-1. Bridge failure will occur at the weakest link of the structure; therefore R/L should be developed for the various components of the bridge (substructure, connections, superstructure, etc.). Each of these components should be considered individually for appropriate retrofit measures and overall bridge retrofit strategy. A cost-effective retrofit option would have a B/C ratio equal to, or greater than, 1.0, where the “Benefit” is the present worth of the avoidable disruption cost (PW) and the “Cost” is the cost of the retrofit measures. If no retrofit strategy is cost-effective, than an acceptance of the risk or the potential replacement of the bridge should be considered.

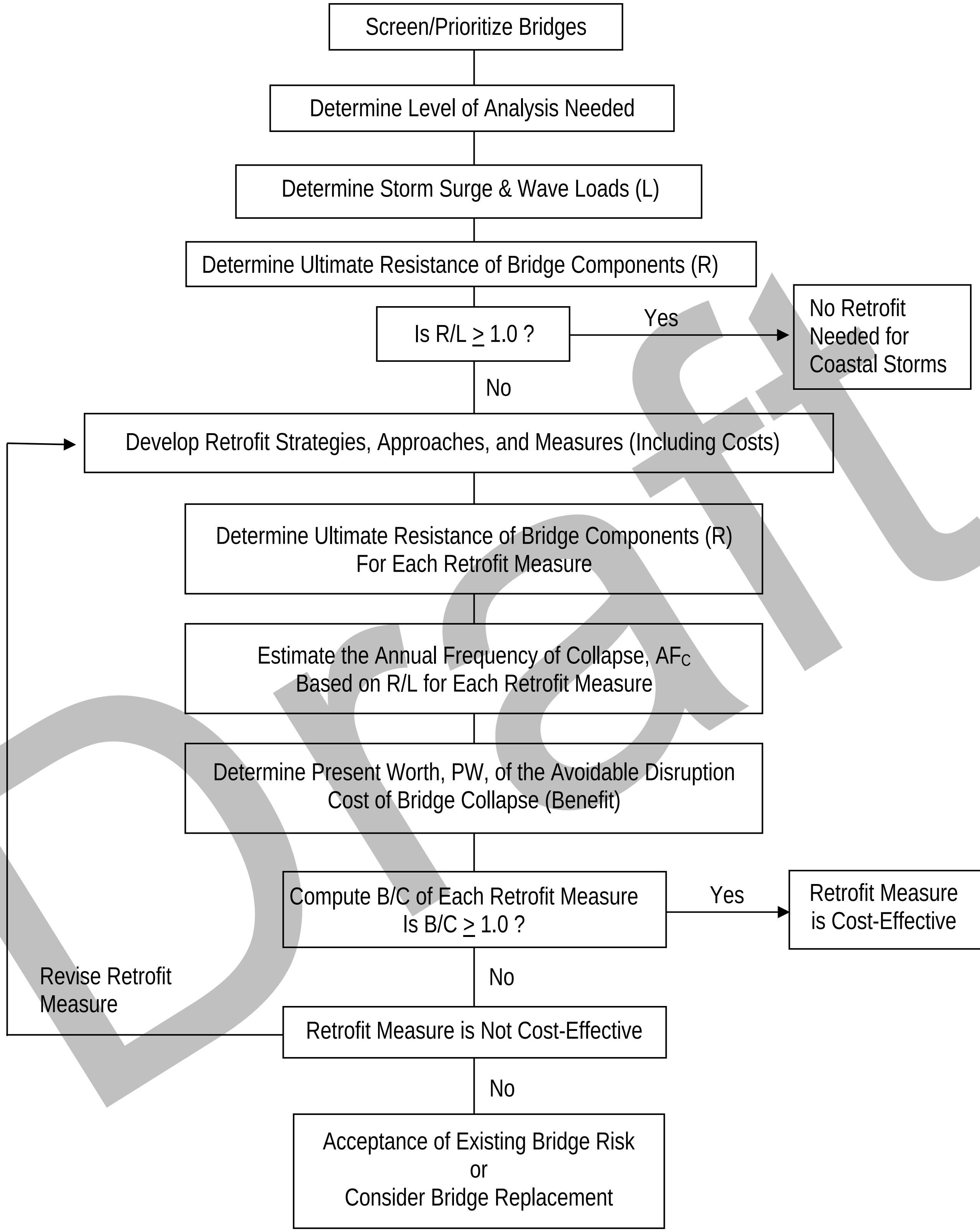


Figure 1.3-1 Flow Chart for Development of Bridge Retrofit Options

2. DEFINITIONS

ASTRONOMICAL TIDE
The tidal levels and character which would result from gravitational effects, e.g. of the Earth, Sun and Moon, without any atmospheric influences.

BATHYMETRY
The measurement of water depths in oceans, seas, and lakes; also information derived from such measurements.

BUOYANCY
The resultant of upward forces, exerted by the water on a submerged or floating body, equal to the weight of the water displaced by this body.

DATUM
Any permanent line, plane or surface used as a reference datum to which elevations are referred.

DEPTH
The vertical distance from a specified datum to the sea floor.

DESIGN STORM
A hypothetical extreme storm whose waves coastal protection structures will often be designed to withstand. The severity of the storm (i.e. return period) is chosen in considering the acceptable level of risk of damage or failure.

DESIGN WAVE CONDITION
Usually an extreme wave condition with a specified return period used for the design of coastal works.

DURATION, MINIMUM
The time necessary for steady-state wave conditions to develop for a given wind velocity over a given fetch length.

EBB CURRENT
The movement of a tidal current away from shore or down a tidal stream.

EBB TIDE
The period of tide between high water and the succeeding low water; a falling tide.

FETCH LENGTH
The horizontal distance (in the direction of the wind) over which a wind generates seas or creates a wind setup.

FETCH-LIMITED
Situation in which wave energy (or wave height) is limited by the size of the wave generation area (fetch).

FLOOD CURRENT
The movement of a tidal current toward the shore or up a tidal stream.

FLOOD TIDE
The period of tide between low water and the succeeding high water; a rising tide. (See figure II-5-16)

HIGHEST ASTRONOMICAL TIDE (HAT)
The highest level of water which can be predicted to occur under any combination of astronomical conditions. This level may not be reached every year.

HINDCASTING
In wave prediction, the retrospective forecasting of waves using measured wind information.

HURRICANE
An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 33.5 m/sec (75 mph or 65 knots) for several minutes or longer at some points. TROPICAL STORM is the term applied if maximum winds are less than 33.5 m/sec but greater than a whole gale (63 mph or 55 knots). The term is used in the Atlantic, Gulf of Mexico, and eastern Pacific.

IRREGULAR WAVES
Waves with random wave periods (and in practice, also heights), which are typical for natural wind-induced waves.

JOINT PROBABILITY
The probability of two (or more) things occurring together.

JOINT RETURN PERIOD
Average period of time between occurrences of a given joint probability event.

MEAN HIGH WATER SPRINGS (MHWS)
The average height of the high water occurring at the time of spring tides.

MEAN HIGHER HIGH WATER (MHHW)
The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN SEA LEVEL (MSL)
The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. MSL is not necessarily equal to MEAN TIDE LEVEL. It is also the average water level that would exist in the absence of tides.

MEAN TIDE LEVEL (MTL)
A plane midway between MEAN HIGH WATER and MEAN LOW WATER. MTL is not necessarily equal to MSL. Also HALF-TIDE LEVEL.

MONOCHROMATIC WAVES
A series of waves generated in a laboratory, each of which has the same length and period.

NUMERICAL MODELING
Refers to analysis of coastal processes using computational models.

OVERTOPPING
Passing of water over the top of a structure as a result of wave runup or surge action.

PARTICLE VELOCITY
The velocity induced by wave motion with which a specific water particle moves within a wave.

PHYSICAL MODELING
Refers to the investigation of coastal or riverine processes using a scaled model.

PROBABILITY
The chance that a prescribed event will occur, represented by a number (p) in the range 0 - 1. It can be estimated empirically from the relative frequency (i.e. the number of times the particular event occurs divided by the total count of all events in the class considered).

REFRACTION (of water waves)
The process by which the direction of a wave moving in shallow water at an angle to the contours is changed: the part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours.

RETURN PERIOD
Average period of time between occurrences of a given event.

RISK ANALYSIS
Assessment of the total risk due to all possible environmental inputs and all possible mechanisms.

SCOUR
Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

SEAS
Waves caused by wind at the place and time of observation.

SHOALING
Decrease in water depth. The transformation of wave profile as they propagate inshore.

SIGNIFICANT WAVE HEIGHT
The average height of the one-third highest waves of a given wave group.

SIGNIFICANT WAVE PERIOD
An arbitrary period generally taken as the period of the one-third highest waves within a given group.

SOUNDING
A measured depth of water. On hydrographic CHARTS, the soundings are adjusted to a specific plane of reference.

STORM SURGE [TO BE REVISED]
A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress. See WIND SETUP.

SWELL
Wind-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch (SEAS).

TIDE
The periodic rising and falling of the water that result from gravitational attraction of the Moon and Sun and other astronomical bodies acting upon the rotating Earth.

TSUNAMI
A long-period water wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave."

UPLIFT
The upward water pressure on the base of a structure or pavement.

WATER DEPTH
Distance between the seabed and the still water level.

WATER LEVEL
Elevation of still water level relative to some datum.

WAVE
A ridge, deformation, or undulation of the surface of a liquid.

WAVE CREST
The highest part of a wave.

WAVE DIRECTION
The direction from which a wave approaches.

WAVE FREQUENCY
The inverse of wave period.

WAVE HEIGHT
The vertical distance between a crest and the preceding trough.

WAVE PEAK FREQUENCY
The inverse of wave peak period.

WAVE PEAK PERIOD
The wave period at which the wave energy spectrum reaches its maximum.

WAVE SETUP
Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

WAVE STEEPNESS
The ratio or wave height to wavelength also known as sea steepness.

WAVE TRANSFORMATION
Change in wave energy due to the action of physical processes.

WAVE TROUGH
The lowest part of a wave form between successive crests. Also that part of a wave below still-water level.

WAVELENGTH
The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WEIBULL DISTRIBUTION
A model probability distribution, commonly used in wave analysis.

WIND SETDOWN
Drop in water level below the still water level on the windward ends of enclosed bodies of water and semi-enclosed bays.

WIND SETUP - LOCAL
On reservoirs and smaller bodies of water, the vertical rise in the still-water level on the leeward side of a body of water caused by wind stresses on the surface of the water.

WIND WAVES
(1) Waves being formed and built up by the wind. (2) Loosely, any wave generated by wind.

3. PAST PERFORMANCE OF COASTAL BRIDGES

3.1 Introduction

The effects of large coastal events on bridges can be broadly grouped into three categories:

- Shifting of spans laterally and longitudinally on the bent caps, in some cases completely off the bent caps. See figures 3.1-1 through 3.1-3
- Damage to girder ends and bent caps from impact of superstructure on substructure. See figure
- Damage to bents from the lateral loads transferred to them. See figure 3.1-5

Other trends that have emerged from past events include the concentration of damage in the lower lying spans. Often spans at higher elevations, to provide navigation clearance or for other reasons, suffered little or no damage while lower elevation spans were significantly damaged or destroyed.

Figure 3.1-1 Dislodged span adjacent to abutment

Figure 3.1-2 Dislodged and collapsed spans

Figure 3.1-3 Low lying spans dislodged and collapsed with surviving high level spans

Figure 3.1-5 Collapsed pier bent with span

One aspect of wave loading that is unlike loads typically experienced by a bridge is the very large vertical uplift forces that develop. Figure 3.1-6 shows the remains of a steel span with a non-composite concrete deck. The vertical loads were sufficient to lift the deck off of the girders and carry it away. Once the deck was removed the vertical forces were greatly reduced, leaving the steel girders in place on the pier bents. It is thought that the vertical force is the driving mechanism behind the displacement of superstructures relative to the piers. In the background of figure 3.1-6 can be seen the pier bents of the concrete spans. All spans have been completely washed away.

Figure 3.1-6 Steel girders missing concrete slab

3.2 Diaphragm-Constraint Air Cavities

The use of full-depth solid diaphragms, both end and intermediate can exacerbate the vertical loads applied during a storm event. These create air pockets between the beams which can retain sufficient air to develop neutral or even positive buoyancy in a superstructure. These pockets contribute to increased vertical loads not only when the superstructure is fully inundated, but the also interact with the wave loads when the static water level is below the deck level. In some instances dislodged spans were found over 200’ away from their original locations after the storm, indicating the superstructure was temporarily afloat. Penetrations through the deck, such as deck drains, can allow air to escape from the interbeam cavities, but these are not normally present in every cavity in a cross-section. See figure 3.2-1

Figure 3.2-1 Air space formed by beams and solid diaphragm

3.3 Superstructure to Substructure Connections

Wave forces, combined with storm surge conditions and set-up, apply large vertical uplift and horizontal loads to a bridge superstructure. The connection of the superstructure to the substructure has proven to be a historical weak point. In many cases, no positive uplift connection is provided to resist the loads that develop. In other cases, the connections provided were inadequate to resist the forces that developed.

One common arrangement of the superstructure to substructure connection for precast prestressed beam superstructures is to utilize elastomeric bearings under the beam ends and steel dowels embedded in the bent cap and extending into the concrete end diaphragms. The primary purpose of the dowels is to provide lateral resistance. A bond breaker is often used on the dowels to allow for future jacking to replace the bearings. This arrangement results in no effective vertical connection between the superstructure and the bent cap. In figure 3.3-1, the vertical dowels can be seen after the span has been dislodged from the bent cap. Note that some dowels remain undamaged and vertical, indicating that the span was lifted enough to dislodge it without damaging the dowels.

Figure 3.3-1 Vertical and bent dowels of dislodged span

Another bearings system that has been used involves a steel and bronze bearings assembly in which the uplift capacity is provided within the bearings themselves. The sole plate is anchored into the beam with embedded straps, and anchor bolts connect the sole plate to the bent caps. In past events these bearings systems have failed at both the sole plate embedment into the beam, and at the anchor bolt connection to the bent cap. See figures 3.3-2 and 3.3-3

Figure 3.3-2 Displaced beam with sole plate still attached

Figure 3.3-3 Sole plate still attached to bearing assembly on bent

Another system used in the past included small steel angles placed alongside the beam flanges and anchored into the flanges and the bent cap. Figure 3.3-4 illustrates this system.

Figure 3.3-4 Clip angle beam restraints

When subjected to the large vertical uplift forces from wave and surge loading, the connections to the flanges were overwhelmed. See figure 3.3-5

Figure 3.3-5 Angle to beam flange connection failed

With the uplift capacity gone, the beams were likely lifted clear of the angles and the superstructure was displaced. See figure 3.3-6

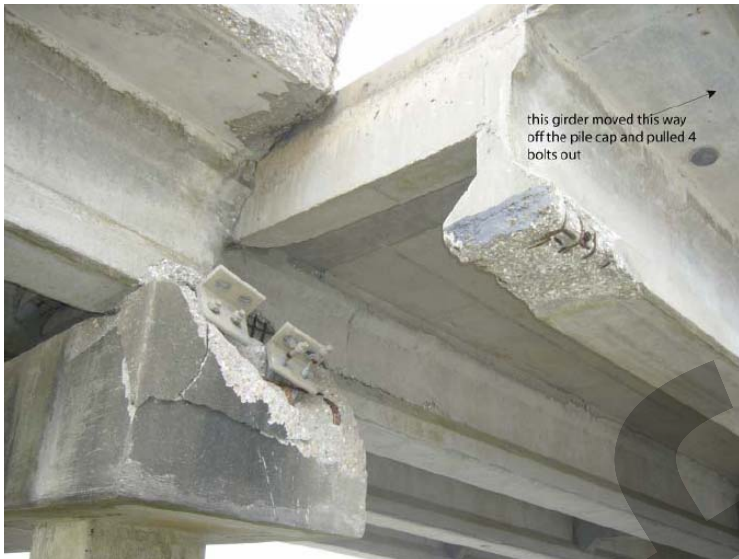


Figure 3.3-6 Failed restraint system and displaced beams

In some cases the lateral capacity of the bearing system was never engaged, as the lack of an uplift capable restraint system allowed the span to move clear of the lateral restraints. Figure 3.3-7 shows a once common restraint system, wherein the steel bearings have pintles which engage a sole plate with slots. The pintles are intended to transfer the lateral loads across the bearings. Any vertical displacement of the span caused by wave and surge loading will disengage the pintles and allow the span to shift.

Figure 3.3-7 Pintle restraint system after loss of span

3.4 Substructure Failure

Should the superstructure and substructure be sufficiently tied together, the failure mode can shift to the substructure. With the ability to transfer the large wave loads to the substructure comes the need to resist those loads. The demands on the bent cap to pile connection, and on the bending capacity of the piles themselves can overwhelm the capacities of these elements. Figure 3.4-1 and 3.1-5 show damage experienced by pier bents when subjected to large wave loads. In figure 3.4-1 the spans were ultimately dislodged before the pier bent collapsed, but after significant spalling occurred in the piles immediately under the bent cap. The bent in figure 3.1-5 experienced a total collapse. It appears that the superstructure, the railing of which can be seen in the water in figure 3.1-5, was still in place at least on one side of the bent at the time of collapse.

Figure 3.4-1 Damaged pier bent

4. SCREENING OF EXISTING BRIDGE STOCKS

4.1 General

The screening procedure provides a method for determining whether a bridge is vulnerable to elevated water levels and wave loading and if further analysis is necessary. This procedure is designed to eliminate non-susceptible bridge spans from further study. The screening procedure is based on an evaluation of only the vertical forces on the superstructure and does not involve a detailed evaluation of the structures resistance to these forces. Horizontal forces and moments are not included in the screening process for simplicity. Resistive forces are based on an estimate of the span's unit weight (weight per unit length of the span). The screening procedure excludes constraints from consideration in the resistive capacity of the span.

Estimating the vulnerability of a bridge to elevated water elevation and wave loading requires a detailed data set describing the study area, including the meteorological and oceanographic parameters (wind velocity, water elevation and wave heights and periods) experienced by the bridge during a design storm event. These parameters along with a detailed description of the bridge superstructure provide the input necessary to estimate the loading on the deck and to establish the span's vulnerability. Note that the screening procedure does not take into consideration load reductions due to site specific conditions such as the span being located on a narrow water body.

Data required for determining design water elevations and wave parameters

- Bridge location and its position in the body of water
- Design wind speed
- Maximum fetch length
- Design storm surge elevation (for coastal bridges)
- Bathymetry – submarine topography and water depth
- Local wind set-up
- Current
- Astronomical tide if not included in the surge data

The recommended procedures for determining design water elevations and wave heights and periods are outlined in Article 6.3.2 of the draft Guide Specifications.

4.2 Bridge Parameters

The bridge parameters required include:

- Low cord elevation
- Span length and width
- Number and height of girders
- Deck thickness
- If readily available, the weight of parapets and median barriers

From these parameters the weight per foot (kips/ft) of the bridge span can be calculated. The weight per foot should include the weight of the girders and deck and where available the additional weight associated with railings, lane barriers, diaphragms, etc. Including the weight of all the bridge components provides results that are more accurate, however neglecting the additional weight is conservative for the purposes of screening. Information regarding the existence of constraints to vertical forces and movements such as anchor bolts or other tie downs, as well as their condition, is usually not readily available and thus are not included as part of the resistive forces in the screening procedure.

4.3 Vulnerability Index

The vulnerability index is the ratio of the estimated design water elevation and wave forces per unit span length divided by the span weight per unit length.

The vertical wave forces consisting of a quasi-static component, F_v, and a vertical slamming force, F_s, may be determined using Articles 6.2.2.2 and 6.2.2.3 of the Guide Specifications. The equations provided in the aforementioned two articles require a wave height, wave length, and a distance from the storm water level to the design wave crest. These parameters, as well as the local wind setup, may be determined based on Article 6.3.2 of the Guide Specifications dealing with a Level I analysis of design parameters.

The vulnerability index is:

$$I_{vulnerability} = \frac{F_s + F_v}{WPF}$$

where:

WPF = dry weight per foot of girders, deck, parapets, barriers, etc

For the purpose of determining F_v and F_s for screening, the wave length peak wave period and maximum wave height may be determined using the equations in Article 6.3.2.4 of the draft Guide Specifications for Bridges Vulnerable to Coastal Storms.

If the vulnerability index is greater than or equal to 0.65 the span is potentially vulnerable to loading due to elevated water level and wave loading. The larger the index value the greater the potential for damage during a design storm event.

4.4 Criticality Index

Another important aspect in the determination of the need for further analysis is the Criticality of the bridge. Factors to be considered in establishing the criticality include:

- the level of social and economic impact
- pre-storm evacuation and post-storm access impacts

- cost of and time required for bridge replacement, etc. if the span is damaged or destroyed.

A number of schemes are being considered for quantifying bridge criticality. The following, relatively simple, method can be used as is or modified and expanded to fit the requirements for a particular location.

Criticality Index	Description
1	Minor impact to economy or emergency needs if closed (alternative routes exist)
2	Medium impact if closed - may lead to a barrier island but an alternative route exists
3	Major impact if closed – only road to a barrier island, evacuation route with no reasonable alternatives
4	Extreme impact if closed – Interstate or major economic connector (detour very long)

The two indices can be used to assist in determining if further analysis is required. Table x, presented below, is an example of a decision matrix for a particular bridge where the action taken would be a function of the two indices. The example in the table exceeds the Vulnerability Index and has a high Criticality Index providing justification for elevating the bridge to the next level of analysis.

Span	Vulnerability Index Number ≥ 0	Criticality Index $1 \leq$ Number ≤ 4	Action Screen or Further Analysis
1	0.65	3	Further Analysis

5. EVALUATION OF EXISTING BRIDGES

5.1 Introduction

There are many choices that have to be made in the evaluation of an existing bridge for retrofit to increase its resistance to coastal storms. These choices include the following which may be interrelated at a given site in a circular, iterative fashion:

- What event or events are to be considered?
- What strategy will be used for a particular bridge?
- Which of the three levels of analysis available in the AASHTO Guide Specifications is to be used?
- What does the cost assessment model of Section 6 of this Handbook indicate is the optimum return on investment for a bridge or group of bridges?

Having determined the above factors, the determination of the forces to be applied follow the procedures in Article 6.2 of the Guide Specifications.

5.2 Selection of Design Event

The AASHTO Guide Specifications are based on a 100-year return period design event. This recurrence interval was chosen because of the ready availability of wind and storm surge data for the 100-year event. It should also be noted that the 100-year event has approximately a 50 percent chance of exceedence (52.9 percent exactly) during the 75-year life of a new bridge. This represents the mean force level over the lifespan of the bridge.

For an existing bridge, the first step is to evaluate the feasibility of retrofitting it (if necessary) to withstand the wave forces presented in Article 6.2.2 of the Guide Specifications. If these retrofits are impractical and/or cost prohibitive, two approaches may be considered by the owner.

The first approach is to simply account for the bridge’s remaining service life and adjusting the design event such that it approximates the mean value during the remaining life of the existing bridge. The following formula may be used to determine the minimum return period event to design for given the remaining service life.

$$\frac{N}{0.69} = RP \qquad (5.2\text{-}1)$$

where:

RP = design event return period
N = remaining service life of the bridge

Meteorological/oceanographic parameters, such as wind speed and surge height, can be adjusted for the return period, corresponding wave parameters, e.g. wave height, period

and height above the storm water level, can be calculated, and wave forces can be determined as specified in Articles 6.2.2 and 6.2.3 in the Guide Specifications.
A second, more rigorous, approach is to perform the optimization routine discussed in the next chapter regarding a Cost Assessment Model. This approach will evaluate the economic merits of retrofit measures by comparing the costs of their implementation with the benefits of increased resistance provided.

5.3 Retrofit Strategies

The following may be considered to reduce the wave forces acting on the superstructure:

- Using open or sacrificial parapets
- Venting the cells that could potentially entrap air creating increased buoyancy forces
- Using large holes in concrete diaphragms or framed cross-frames and end diaphragms on concrete superstructures to promote venting and the exchange of trapped air between spans
- Using continuous superstructures to increase the reactive force of individual spans
- Using solid or voided slab bridges to reduce buoyancy forces in low lying spans

A variety of retrofit options intended to implement these basic approaches are discussed further in Section 7 of this Handbook. Schematic sketches are also included as a starting point for considering appropriate site-specific details.

5.4 Levels of Analysis

5.4.1 Introduction

The Guide Specification recognizes three levels of analysis of increasing sophistication, but presumably more accurate assessment of the wave height, surge height, wind speed, and local wind setup. The increased accuracy of the higher levels of analysis are obtained through the use of sophisticated computer modeling which requires expertise usually associated with coastal engineers rather than bridge engineers.

5.4.2 Level I Analysis of Design Parameters

The storm surge, wind setup, current, wave height and period used in a Level I analysis of a given storm event are assumed to occur simultaneously. The joint probability of all of these contributing events being maximized at the same time is not considered in a Level I analysis, and, therefore, the results should be conservative, possibly quite conservative.

A Level I analysis:

- requires the least effort of the three levels to perform,
- is the most conservative in the magnitude of the predicted forces, and
- is for the most part, based on readily available information.

The Level I analysis is designed to be conservative due to the lower confidence levels associated with the input parameters for computing design water levels and wave heights and periods. The same year (given year) values are used for all the components that make up the design water elevation and the wave parameters. For some situations (e.g., open coast, center of a near circular bay) this combination will produce the given year event. However, for most bridge locations (e.g., bridges over long narrow waterways) the combination of given year components will yield a less frequent event. These differences are addressed in the load modifiers presented in tables 6.2.2.6-1 and 6.2.2.6-2 of the Guide Specification.

A Level I analysis is one step above a screening analysis that might be used to identify critical bridges for retrofit, and the approach is suitable for eliminating bridge spans from further analysis. In most cases, a Level II analysis should be performed prior to retrofitting.

The information described herein for a Level I analysis could lead to a false sense of confidence regarding the ability for engineers without a coastal background to correctly assess a given situation. Even for a Level I analysis, a review of data and interpretation of results by a coastal engineer is required.

A Level I analysis for the determination of maximum wave crest elevation should include consideration of the following:

- Bridge location
- 100 year design wind speed
- Maximum fetch length and orientation relative to the open coastline
- 100 year storm surge elevation and the mechanisms considered in its determination
- Bathymetry – submarine topography

The Guide Specification permits the use of empirical equations from the *Shore Protection Manual* to determine wave height and period. Other empirical equations are used to produce the maximum wave height, etc.

5.4.3 Level II Analysis of Design Parameters

A Level II analysis may be used to improve upon any of the data or analytical techniques/equations used in Level I.

The primary difference between Level I and Level II analyses is the accuracy of the information used to compute the design water elevations and wave parameters. Depending on the circumstances, a Level II analysis may be performed initially or following a Level I analysis. Where a Level I analysis has preceded Level II, all quantities used to compute design water elevations and the wave parameters in the Level I analysis should be reassessed, and those deemed improvable should be reevaluated.

A Level II analysis:

- requires more effort than a Level I analysis,
- is more accurate than a Level I analysis, and

The Level II analysis allows for a wide range of possible improvements compared to a Level I analysis. Additional or more recent measurements may be required for such quantities as bathymetry. Computer models will most likely be needed for reliable estimates of wind setup and wave parameters.

In some situations a Level II analysis may be cost-justifiable as the minimum level of effort required to obtain the information needed to retrofit an existing bridge. This is particularly true of long low-lying bridges with numerous spans.

5.4.4 Level III Analysis of Design Parameters

A Level III analysis may be used to determine design parameters for bridges critical to regional economy, safety, or rescue and recovery operations for bridges where substantial repair and/or replacement costs may be incurred if damaged by a coastal storm event. Where sufficient meteorological and oceanographic data exists to consider return periods other than 100 years, a multi-level approach with site-specific performance criteria may be considered.

Level III analyses:

- are more time consuming and costly to perform;
- produce more accurate results than Levels I and II analyses,
- account for the joint probability of the various design parameters; and
- are necessary for large and/or important bridges deemed susceptible to storm surge and wave loading.

The modeling effort improves the accuracy of all meteorological/oceanographic parameters needed for the computation of storm surge and wave loading on bridge sub and super structures. This improved accuracy includes design water elevations, current velocities, currents, and wave heights and periods and their joint probabilities.

A Level III analysis requires an extensive computer modeling and analysis effort and possibly the measurement of bathymetry, as opposed to use of published information, and model calibration parameters such as water elevations and waves.

There are a number of numerical models for computing hurricane generated wind fields, storm surge hydraulics (water elevation, depth averaged current velocity), and wave parameters in use, each with their strengths and weaknesses. The following procedure is one that has been successfully used and can be considered as a guideline for performing a Level III analysis:

1. Perform hurricane wind and pressure field hindcast for as many hurricanes (or significant storms) as have impacted the area of interest as time and resources allow.
2. Perform storm surge and wave hindcasts using coupled wave and surge models for hurricanes using the hindcasted wind and pressure fields.
3. Using the water elevation and wave information at the bridge site for each of the hindcasted storms, perform extremal analyses on these parameters to obtain the desired design water levels and wave conditions.
4. Knowing the design water levels, wave conditions and structure parameters compute the storm surge and wave induced forces and moments on the bridge spans.

6. COST ASSESSMENT

Cost-effectiveness can be used to evaluate and determine appropriate retrofit strategies for existing coastal bridges exposed to potentially catastrophic damage or collapse due to storm events (storm surge plus wave forces). This cost assessment procedure would used to evaluate existing bridges where retrofit using the design criteria outlined in the Guide Specifications could not be fulfilled due to unreasonable or prohibitively high costs. For those situations, the economics associated with the cost-effectiveness of risk reduction can be used to determine the optimum retrofit measures. It should be noted that the cost assessment procedure does not attempt to address the issue of what is an appropriate level of safety for an existing bridge; its focus is on the ratio of benefits to cost.

The choice of retrofit measures would be based on a cost-effectiveness acceptance criterion, such as a benefit cost analysis, where the cost of strengthening the bridge, or selected components (such as the substructure, superstructure, connections, etc) is compared to the benefits of risk reduction. It is recommended that the analysis methodology used to test economic feasibility be a conventional benefit/cost (B/C) ratio calculation in which the present worth of the disruption cost, PW, for each year of the anticipated bridge life is compared against the total present worth of the costs to replace and maintain the bridge structure, or the retrofit and bridge strengthening measures required to provide those benefits. The present worth of the costs and benefits should be computed over a specific time period (usually the life, or the remaining life, of the bridge) in order to identify incremental costs and benefits attributable to the bridge strengthening. The present worth is the cumulative present value of a series of costs and benefits occurring over time, and is derived by applying to each cost or benefit in the time series, an appropriate discount factor, which converts each cost or benefit to a present value. This insures that all costs, benefits, and other values are expressed in constant dollars. Growth of the disruption cost over time should be considered in the analysis (for example, a growth in ADT over time would increase motorist detour and inconvenience costs in the event of a bridge collapse).

The approximate benefits used to compare against the cost of strengthening or retrofitting the bridge can be estimated as follows:

$$PW = AF_C\,(DC)\,[(1+g) / (i-g)]\,\{1 - [(1+g)/(1+i)]^N\}$$

where:

PW = present worth of the disruption cost
DC = disruption cost associated with bridge collapse
AF_C = annual frequency of bridge collapse
i = real discount rate
g = annual rate of growth of the disruption costs.
N = design life of bridge

The disruption cost (DC) associated with bridge collapse can be computed as:

$$DC = PRC + SRC + MIC$$

where:

PRC = bridge substructure replacement/repair costs
SRC = bridge superstructure replacement/repair costs
MIC = motorist inconvenience costs

Additional costs such as loss of life, environmental, business, social, and other disruption costs may often be incurred in a catastrophic bridge collapse; however, since these costs are usually subjective and difficult to estimate, they would not be quantitatively included in the analysis for computing DC; however, their potential impacts should be qualitatively considered in the evaluation.

Substructure replacement costs (PRC) and superstructure replacement costs (SRC) are those costs associated with the replacement or repair of bridge piers and spans damaged by a given storm event. Depending on the storm event, waterway environment and bridge location, this may include the entire bridge from shoreline to shoreline, or only a portion of the total bridge length. Depending on the retrofit measure being studied, it may be the superstructure replacement costs only (for example, if the superstructure is allowed to fail while the substructure remains undamaged). For bridges with a high level of continuity, damage to one pier or span component may require the repair/replacement of portions of the bridge structure located relatively far away from the collapse location. A significant key to the B/C analysis is an estimate of the length of time of the bridge outage needed to repair or replace the damaged structure.

Motorist inconvenience costs (MIC) include costs incurred by motorists who would be forced to use a detour route for the bridge outage period. It also includes toll revenues lost by the out-of-service facility owner, if it is a toll bridge. Estimates of MIC require the identification of detour routes, collection of traffic volume data, and the calculation of incremental vehicle operating costs using prescribed AASHTO standard procedures. In many cases the MIC costs are quite large (often orders of magnitude higher then PRC and SRC), particularly if there are no nearby alternative routes, or if the bridge outage time is lengthy.

The discount rate (i) is used to bring back future costs and benefits to present value. For future costs and benefits calculated in constant dollars, only the real cost of capital should be represented in the discount rate.

The rate of growth of disruption costs (g) accounts for increasing motorist traffic on the bridge due to growth in the general region and local communities. The influence on g for motorist traffic can be computed using future ADT volumes estimated for the bridge.

Traditionally, cost-effectiveness is indicated by a B/C ration greater than 1.0. A typical relationship between risk and the cost of risk reduction is shown in the figure below.

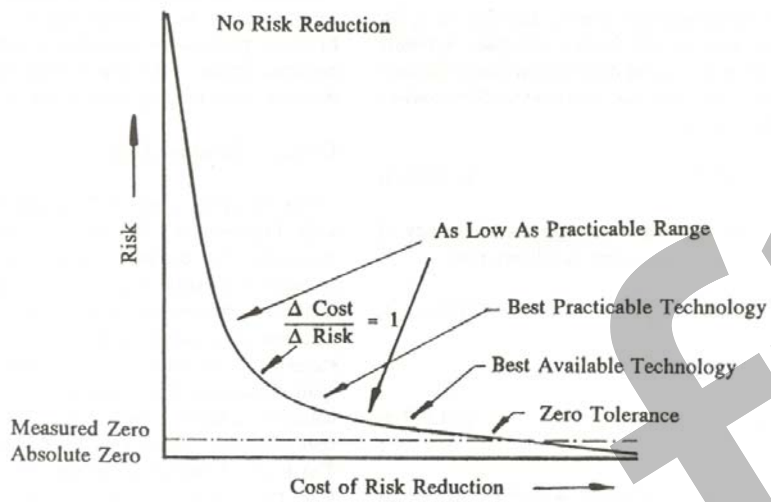


Figure 6-1 Cost-Effectiveness of Risk Reduction

For the storm surge and wave impact evaluation of bridges, a key element is the accurate estimation of the annual frequency of bridge collapse (AF_C) based on the design event and the remaining life of the bridge. The estimation of AF_C should consider not just the risk associated with the storm event occurring at the bridge site, but also the probability of failure of bridge components based on the surge and wave forces, and the ultimate resistance strength of the bridge components.

AF_c can be calculated as:

$$AF_C = \left\{1 - \left[1 - P_{f75}\right]^{\frac{N}{75}}\right\} \times \frac{1}{N}$$

where:

P_{f75} = approximate probability of failure during a 75 year design life, taken from figure 6-2

N = remaining life of existing bridge

To find P_{f75}, the ratio of the factored resistance of the existing bridge to the factored calculated loads (L) based on a 100 year design event is first calculated. Figure 6-2 is then entered with this ratio and P_{f75} can be read from the y axis. For failure modes involving forces acting against the dead load of the bridge, the dead load should be added to the resistance portion of the ratio.

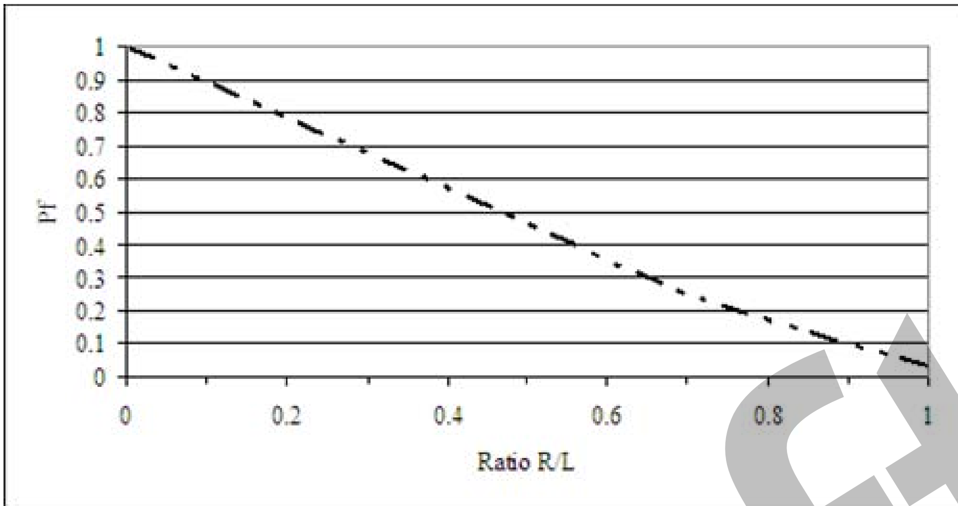


Figure 6-2 Pf given as a function of resistance to load ratio

Sample Problem

An existing bridge has a 50 year remaining life. Two damage scenarios and two corresponding retrofit options are being considered:

1) Bridge is totally destroyed (substructure and superstructure) by the 100-Year storm event, and

2) Bridge is partially destroyed (the substructure survives but the superstructure is allowed to fail)

Based on the storm surge – wave force/pier resistance strength ratio, it was determined that R/L = 0.8 for all substructure and superstructure components. From figure 6-2, this results in a Pf equal to 0.17

$$AF_C = \left\{1 - \left[1 - P_{f\,75}\right]^{\frac{N}{75}}\right\} \times \frac{1}{N} = \left\{1 - \left[1 - 0.17\right]^{\frac{50}{75}}\right\} \times \frac{1}{50} = 0.0023$$

For the bridge the bridge substructure replacement/repair costs, PRC, were determined to be \$10 million; the bridge superstructure replacement/repair costs, SRC, were determined to be \$35 million; and the motorist inconvenience costs, MIC, were determined to be \$10 million per month for every month that the bridge was out of service (based on a high ADT and a long detour to the next bridge crossing of the waterway).

Assume a typical discount rate of i = 0.04 (4 percent).

Assume a annual ADT growth rate of g = 0.005 (0.5 percent per year)

Scenario 1: Bridge is totally destroyed (substructure and superstructure)

If totally destroyed, it was estimated that it would take 18 months to replace the bridge on an emergency basis. The disruption cost (DC) associated with bridge collapse can be computed as:

$$DC = PRC + SRC + MIC = \$10 + \$35 + (\$10/mo)(18\ mo) = \$225\ million$$

The present worth of the disruption cost would be:

$$PW = AF_C\,(DC)\ [(1+g) / (i-g)]\ \{1 - [(1+g)/(1+i)]^N\}$$

$$PW = (0.0023)(225)[(1+0.005)/(0.04–0.005)]\{1–[(1+0.005)/(1+0.04)]^{50}\}=\$12.2\ mil$$

The retrofit option for the bridge substructure has been estimated to cost $5 million, and the retrofit option for the bridge superstructure has been estimated to cost $25 million; therefore, the total cost of retrofitting the entire bridge equals $30 million.

In determining the cost-effectiveness, the benefit, B, is avoiding the disruption cost; because the retrofit will avoid all disruption costs, therefore, B = PW. The cost, C, is the cost of the retrofit option, therefore:

$$B/C = \$12.2/\$30 = 0.41$$

Since the B/C is less than 1.0, the retrofit option is not cost-effective.

Scenario 2: Bridge is partially destroyed

Under this scenario, the piers would be retrofitted to survive, but the superstructure would be allowed to fail. It was estimated that it would take only 8 months to replace just the superstructure on an emergency basis. The disruption cost (DC) avoided by this rerofit can be computed as:

$$DC = PRC + SRC + MIC = \$10 + \$0 + (\$10/mo)(18\text{-}8\ mo) = \$110\ million$$

The present worth of the disruption cost would be:

$$PW = AF_C\,(DC)\ [(1+g)/(i-g)]\ \{1 - [(1+g)/(1+i)]^N\}$$

$$PW = (0.0023)(125)[(1+0.005)/(0.04–0.005)]\{1–[(1+0.005)/(1+0.04)]^{50}\}=\$5.91\ mil$$

The retrofit option for the bridge piers has been estimated to cost $5 million, therefore:

$$B/C = \$5.91/\$5 = 1.18$$

Since the B/C is greater than 1.0, the retrofit option is cost-effective and should be considered for implementation.

Optimization of B/C Ratio

Another alternative approach to the retrofit of existing bridges (or the design of a new bridge) involves optimizing the benefit cost ratio. While the B/C ratio may be greater than 1.0, it may still not be economical to design the retrofits to withstand the 100-year design storm event. On the other hand, an under-designed structure will suffer frequent damage and the disruption costs may be prohibitively expensive. The optimal structure design should balance initial and long-term costs.

In addition to the above procedure of evaluating the cost effectiveness of each considered retrofit approach/measure without explicit consideration of the resulting effective design force level, the same tools outlined in this section can instead be used to the same end by setting a target resistance based on a reduced design level (return period), and then selecting retrofit measures to achieve the target resistance. The B/C ratio can then be calculated, and the procedure repeated for various target return periods until an optimum B/C ratio is reached. In either case the object is to determine the economically optimum retrofit, albeit with an increased level of risk over that required by the Guide Specification.

7. RETROFIT STRATEGIES, APPROACHES AND MEASURES

7.1 Introduction

It is the intent of this section to present retrofit concepts that may be used to make vulnerable bridges more resistant to coastal storms. A retrofit can be broken down into three levels, the Strategy, Approach, and Measures. The retrofit Strategy is the overall plan for addressing the vulnerability of the bridge to coastal storm events. A strategy may be composed of one or more approaches. An approach is a specific philosophy used to improve the performance of the bridge. Each approach consists of implemented measures which enact the approach philosophy. For example, an approach may be to strengthen the substructure and would consist of measures such as adding additional piles and lengthening the pile cap or jacketing the pile cap to pile connection.

Each bridge will present a unique set of vulnerabilities and constraints, and each project must be approached individually. It is entirely possible that the best strategy for a bridge will be to do nothing to the existing bridge, and plan for its eventual replacement after a coastal event. Factors such as the cost of the needed retrofit measures, the remaining useful life of the bridge, and the bridge's place in the transportation infrastructure must be weighed in any decision process on retrofitting a coastal bridge.

Several retrofit approaches that might be implemented are listed below. Many other approaches could be considered, and for any specific bridge the best approach may not be in this list. However, these were determined to be the most applicable of those considered. For other approaches and measures not adopted, see the project report.

At this writing, some of the retrofit approaches are more quantifiable than others either by the nature of the retrofit or by the provisions of the Guide Specifications. For example:

- Currently Quantifiable
  - Reduction of buoyancy loads
  - Reduction of wave loads
  - Strengthening connection of superstructure to substructure
  - Strengthening the structural capacity of substructure
  - Strengthening the geotechnical capacity of substructure
  - Accepting loss of superstructure to protect substructure
- More Qualitative
  - Connection of adjacent spans

A review of the force equations in the Guide Specification will show that for bridges with a bottom of structure elevation significantly below the top of wave crest elevation (>2 feet below), the vertical forces can become large enough to preclude any strategy that includes resisting the forces by structural means. In these cases, it is likely the best strategy would be to utilize venting to reduce the forces, provide tie-downs only to prevent unseating from lesser storm events, and plan for the loss of the superstructure while protecting the substructure. When determining the required strength of the tie-downs, care should be taken not to exceed either the strength of the superstructure in negative bending at midspan, or the strength of the substructure and foundation elements.

7.1.1 Reduction of Buoyancy Loads

The configuration of many bridges creates the potential for air to become trapped under the deck when water rises above the bottom of the girders. This trapped air creates a buoyancy force that may increase the uplift force on a bridge during a surge or wave event. The entrapment of air, which occurs primarily on bridges with concrete decks and solid diaphragms, may be reduced by providing vents through the deck or diaphragms, or by replacing solid diaphragms with steel frame diaphragms.

There are two conditions for which the release of trapped air is desirable: when a span is completely inundated where the air is trapped by the rising storm water level, and when a span is within the wave height zone where air is trapped by the fluctuating wave surfaces. For the first case, the rise in static water level is usually a slow process, and small openings are all that is required to evacuate the air. A path should be provided to vent the air to the outside. For the second case, the air needs to be evacuated in a relatively short time period, on the order of a half to one second. This requires a relatively large area of openings to vent the air. However, as the correlation of the wave arrival along the span length of the bridge is not expected to be perfect, allowing the air to move longitudinally along the bridge by way of openings in the diaphragms may provide all the relief that is necessary.

Buoyancy forces from static water level rise have the potential to be about the same magnitude as the dead load of a bridge span. Several bridges impacted by Hurricane Katrina were investigated to determine the potential effects of buoyancy. It was assumed that the superstructures were submerged in static seawater (unit weight = 64 lb/ft^3) up to the level of the top of the sidewalk or top of the deck if no sidewalk was present. The air trapped under the deck was assumed to compress according to the ideal gas law as the water level exceeds the deck level. Deck drains, when present, were assumed effective at permitting air to escape, so cavities containing deck drains were assumed to be filled with water. The US-90 Biloxi Bay Bridge (1959 design) used 52’ spans for much of the bridge. Each 52’ span used six prestressed beams that were 3’0” deep and were spaced at 6’0” to 6’5” center to center. Solid diaphragms extended from the deck to 6” above the bottom of the beams. Deck drains were present in one of the air cavities. The buoyancy load on a span due static water up to the level of the top of the sidewalk equaled 86 percent of the span’s dead load. The I-10 Lake Ponchartrain Bridge (1960 design) used 65’ spans for much of the bridge. Each 65’ span used six prestressed beams spaced at 7’7” center to center. The depth of the section from the bottom of the girder to the top of the deck was about 4’ 6”. Solid diaphragms extended from the bottom of the deck to about 13” from the bottom of the beam. Provided deck drains did not extend into an air cavity. The buoyancy load on a span due to static water up to the top of the deck equaled 104 percent of the dead load of the span, meaning the section would float if it were unrestrained. These observations do not even include the vertical force from the wave.

(Note that when vertical wave force, F_v, is calculated using Article 6… of the Guide Specification the buoyancy is included so that the vertical resistance is the dry weight of the structure.)

The number and size of vents required must be determined based upon the area and depth of the air cavity, the permissible differential water level (difference in elevation between the water in the cavity and the water outside of the cavity), and the maximum time allowed for air evacuation. Preliminary calculations equated the maximum permissible differential water level to a maximum pressure in the air cavity. This pressure was then used to determine an exit velocity for the pressurized air. Knowing the exit velocity of the air, the depth of the cavity, and the maximum time allowed for air to escape, it is possible to determine the required area of vents as a percentage of the horizontal area of the air cavity. The maximum time allowed for air evacuation should be based on the time it takes for water outside of the cavity to rise from the bottom of the cavity to the top of the cavity. For waves, this time would be on the order of seconds or fractions of seconds, for surge this time would likely be on the order of many minutes or even hours. Preliminary calculations show that to evacuate a 4’ deep air cavity in 3 seconds while limiting the differential water level to 1’, vents with an area equal to 0.58 percent of the area of the cavity would be required. This means an air cavity bounded by girders and diaphragms with an area of 75 square feet (5’ spacing by 15’) would require five 4” diameter holes. A span containing 16 of these cavities (for example a 60’ span with 5 girders) would require 80 holes. To evacuate the cavity in 1 second, three times the number of holes would be required. This indicates that vents may not be practical for significantly reducing buoyancy forces during short duration events (waves). However, the reduction of buoyancy loads during longer duration events (surge) may be accomplished using vents with a relatively small total area. It is important to note that the use of several smaller diameter vents instead of a single larger vent per air cavity may be advantageous for structural and constructability reasons.

Vents which require drilling or coring of existing components should be designed and installed in a manner that will minimize the possibility of adverse effects on the structure. Vents should be placed to clear reinforcing whenever possible, and the effects of accidental damage to reinforcing should be investigated before installation is performed.

7.1.2 Reduction of Wave Loads

The action of waves hitting the deck and girders of bridges leads to significant forces being applied to the structure, both horizontally and vertically. In some cases it may be possible to reduce these forces. This may be done by altering the waves hitting the bridge (through the use of artificial reefs, for example), changing the cross section that the waves strike, or by raising the cross section so that it is not impacted by waves. This approach is likely to be more costly than other approaches, but may prove advantageous for specific cases.

7.1.3 Connection of Adjacent Spans

Observation of surge and wave damage has revealed that continuous spans may experience less shifting (lateral and longitudinal displacement) during storm surge and wave events than do simple spans. It is thought that this may be due to the correlation effects of wave impact along the length of the bridge. When one span of a bridge is experiencing maximum wave/surge loading, adjacent spans will experience loads with a smaller magnitude due to the small probability that a wave will impact the spans at the same time. For continuous spans the section experiencing maximum loading will have the benefit of the adjacent span's reaction at the bearings, preventing unseating and consequent movement of the spans. Connecting spans may permit a span experiencing maximum loading to engage the dead load resistance of adjacent spans which are experiencing loads of a smaller magnitude.

Connection of adjacent simple spans can be accomplished by connecting the webs of the beam ends or by connecting the end diaphragms. The purpose of the connection is to share dead load under uplift conditions, and not to transmit loads under normal operating conditions. A connection that requires some limited amount of movement before the connection is engaged will likely prove effective.

Connecting adjacent spans as a retrofit for wave loads will generally only be suitable when the surge/wave loads on a span are not significantly higher than the dead load resistance of the span. If the surge/wave loads are significantly higher, adequate reserve capacity of adjacent spans may not be available and spans may be shifted or lost. Knowledge of the sea state (spacing of waves, direction of waves, variation of wave height perpendicular to the direction of wave travel, etc.) and engineering judgment will be required to determine the likelihood of simultaneous loading of adjacent spans and how much additional resistance adjacent spans will provide.

7.1.4 Strengthening the Connection between Super and Substructures

Surge and wave damage to highway bridges in past events has consisted of displacement of superstructures, both laterally and longitudinally, on the pier bents and in some cases the displacements were large enough to completely dislodge the spans. In some cases spans appear to have been lifted above the shear blocks or angles that were provided to supply lateral restraint and then displaced laterally. Tying the superstructure to the substructure may provide suitable means of preventing the shifting of spans due to uplift and lateral loads during surge/wave events

Any connection retrofit measure should account for the normal displacements that occur between the superstructure and substructure. Movements due to the following should be provided for:

- Thermal expansion and contraction of girders
- Live load rotation of girders
- Vertical deformation of elastomeric bearing pads under live loads

In addition to these displacements, the normal maintenance needs of a bridge should also be provided for, jacking access for bearing replacement being one example.

As with any structural modification, the loads need to be followed through the structural load path to ground. When utilizing this retrofit approach, the connection of the superstructure to the bent cap may create additional failure modes:

- Negative bending in the superstructure at midspan due to the vertical uplift forces along the span and the restraint reactions
- Shear at the ends of the girders due to the same
- Reduction in bending capacity of the piles due to decreased compression, or even tensile forces
- Increased lateral loads on the substructure

Preliminary indications have shown that for coastal bridges with the largest exposure vulnerabilities, existing substructures will not be capable of resisting the large horizontal loads expected. More than any other approach, care must be exercised in the application of this concept that potentially undesirable consequences arising from its implementation are accounted for.

Depending on the magnitude of the wave/surge forces expected to act on an individual bridge, utilization of this approach in isolation may not be sufficient, and may in fact result in more damage to the bridge than if left unretrofitted. Utilization of fuse elements in the restraint system, in conjunction with a realistic expectation of damage may provide the best overall retrofit strategy.

7.1.5 Strengthening Substructure

The lateral loads and vertical uplift that can be caused by wave/surge loading were likely not included in the original designs for substructure units. The magnitudes of these loads can be many times greater than the original design loads. Aspects of the substructure that have not performed well in past events include the connection between the pile and the pile cap (especially if precast elements were used for both), and the bending strength of the pile immediately below the pile cap.

This approach may be needed if the superstructure is well connected to the bent cap, or to ensure that failure will occur at someplace other than the substructure.

7.1.6 Strengthening Geotechnical Capacity of Foundation

This approach is useful if the wave/surge loads, after transmission through a structurally sufficient load path, overload the existing foundations. In general efforts to improve the capacity of foundations tend to be costly, and this approach should only be implemented after a careful study of alternatives.

Retrofit measures which implement this approach can be divided into two general groups: auxiliary foundations, and soil improvement. Both measures are difficult to implement on an existing structure, but can be useful in completing the load path to ground.

7.1.7 Accepting Loss of Superstructure to Protect Substructure

Depending on the cost assessment results, and various other programmatic and financial factors, the best approach to retrofitting a coastal bridge may be to allow the superstructure to be lost during a storm event in order to protect the substructure. In past events, the presence of an intact substructure has greatly reduced the time and cost required to put a bridge back in service, compared to a complete replacement of both super and substructures.

There are some measures that can be taken to improve the performance of this approach. If for the 100 year storm it is decided to sacrifice the superstructure, consideration should be given to the performance of the bridge in lesser events. Utilization of fuse elements to keep the superstructure in place for more frequent storms, but allow its loss in a large event should be considered. Additionally, measures which limit the damage to the bend cap during the ratcheting displacement of the superstructure across it should be considered also as part of this approach.

7.2 Retrofit Measures

Due to the large variation in bridge details, the retrofit measures presented should be considered as general rather than specific information. Sketches illustrating retrofit measures are intended to convey the intent of the strategy involved, not specific instructions as to how the retrofit should be proportioned or designed. Analysis issues will vary depending on circumstance, and although suggestions of issues requiring investigation are given, these lists should not be considered comprehensive. When the load paths or boundary conditions of a structure are altered by a retrofit, affected elements must be analyzed to ensure that they will function as intended. Members or connections that do not have adequate capacity to function in the retrofit strategy should be strengthened, or an alternative measure should be investigated.

The measures shown are intended to be passive under typical operating conditions, unless otherwise stated. Generally, retrofits should not transmit loads due to typical structural behavior such as live load rotation or thermal expansion. This will reduce the possibility that retrofits will cause unanticipated damage to the structure. In many cases eliminating load transmission under service loads will necessitate connections that require some differential movement before they are engaged. The movement required to engage the connection may, in some cases, result in impact loads on the connection. These loads have the potential to be significantly higher than if the connections were engaged without movement. This needs to be accounted for when determining adequate clearances.

In many situations, multiple elements of a restraint system are intended to operate in parallel, i.e. become engaged at the same displacement. In order for this to occur, attention should be paid to construction tolerances and the use of shims or other methods of adjusting tolerances in the field. If this is not practical for a given application, the non-uniform distribution of load to each individual element should be considered. This issue may be particularly problematic if structural fuses are utilized as part of the retrofit.

Attention should be paid to the maintenance requirements of the chosen retrofit measures. Complex mechanisms which may require frequent inspection or repair should be avoided. The materials used should be durable and able to withstand prolonged marine exposure. If traditional structural steel is used, protective measures such as galvanizing, metalizing, or epoxy coating may be desirable. The use of weathering steel in marine environments is not recommended.

7.2.1 Cored Deck Vents

Coring holes in the bridge deck provides venting for the air space between the beams below. Installation of cored deck vents can be accomplished from above, without special access methods. Temporary lane closures may be required to allow vent installation for all airspaces.

Cored deck vents have the disadvantage of creating holes in the wheel load sensitive part of the superstructure. It is likely deck vent installation will require the severing of deck reinforcing bars, which will have a negative impact on the deck capacity. Consideration should be given to the effect of vents on the ridability of the deck, especially in regards to any increase in impact loads. Decreased deck smoothness may be mitigated by locating vents near the center of lanes and providing covers over the vents. Covers can be grid type to allow free air flow, or blow-out types.

Depending on the size and number of deck vents, they can be used to evacuate air for the static water level rise condition and also for the wave induced air entrapment condition.

7.2.2 Diaphragm Vents

Diaphragm vents are primarily intended to relieve the wave-induced entrapped air buoyancy effect. The area provided by the vents may need to be large, on the order of ?? square feet, in order to evacuate the air in a short enough period of time. They have the advantage of not creating holes in the deck and not requiring temporary lane closures. However, they will require access to the under side of the superstructure.

7.2.3 Replacement of Solid Diaphragms

This measure is also intended to allow the entrapped air to move longitudinally along the bridge. The construction effort required will be greater than the installation of cored vents, but it will provide a much larger opening for the air to pass through. The concrete diaphragms would be removed and replaced with steel truss-type diaphragms where required. Consideration should be given to removing diaphragms completely, where allowed by the LRFD Bridge Design Specifications.

Details for this measure would include the saw-cutting of the concrete diaphragms, and the anchoring of steel frames in their place. Care must be taken to ensure a secure embedment of the steel frame anchorage.

7.2.4 Break-away Barriers

One cross section modification is the use of breakaway bridge barriers. The barriers are designed to breakaway when they are impacted by waves from the back side, but not when impacted by vehicles from the front face. This reduces the vertical surface that waves may strike. The reduction in area will reduce the horizontal loads imparted on the structure. This may be a useful measure if the effects of the vertical loads have been mitigated, but the magnitude of the horizontal loads are still problematic. Consideration should be given to the secondary damage likely to occur from loose sections of barrier as they are moved around on the deck by the waves.

7.2.5 Structural Depth Reduction

Another cross sectional modification is the replacement of I-girder spans with slab spans or adjacent box beams. This modification will reduce the depth of the structure and hence horizontal loads on the superstructure. Experiments have shown that, in addition to the horizontal wave force applied to the waveward exterior girder, a significant horizontal wave force is also applied to the vertical surfaces of interior girders. Slab spans and adjacent box beams do not have interior girders with exposed vertical faces, thus the interior girder loads will be eliminated. The overall depth of slab spans or adjacent boxes should be smaller than the girders that they are replacing. This reduced height will reduce the vertical surface area of the fascia, and thus further reduce the horizontal wave force on the superstructure. If spans with reduced depths are used, the profile grade of the roadway should be maintained. This will require the beam seats to be raised an appropriate distance, but will raise the bottom of girder elevation, which may lead to an additional reduction in wave loads.

This measure will likely be very expensive, as it essentially entails the complete replacement of the superstructure. Consideration should be given to raising the superstructure in conjunction with, or in place of, this measure.

7.2.6 Artificial Reef

Changing the waves that impact a bridge is a significant task and requires expertise outside the scope of structural engineering. Significant environmental issues would need to be addressed prior to implementing this measure. It is not likely this measure will be practical in most cases, but there may be several instances where it is appropriate.

7.2.7 Raising Superstructure

Of all the retrofit measures presented in this manual, the raising of the superstructure has the greatest potential of avoiding all damage from coastal events. Raising the superstructure of a bridge so that it is not impacted by waves would eliminate wave forces on the superstructure. Even if raising the superstructure completely above the top of wave elevation is not possible, reduction of wave forces may be achieved.

A number of challenges are presented by this method which must be addressed prior to implementation:

- Construction staging to maintain traffic during construction
- Increased overturning moments on the substructure from wind and/or wave/surge
- Increased dead loads from additional substructure
- Increased grades and roadway geometric issues at the ends of the bridge

It may be possible to perform this retrofit under traffic if a construction ramp or bridging unit that is advanced with construction is used. The unit would allow the roadway to transition between the new and existing elevations. The additional elevation of the bridge must be transitioned to existing grade in an appropriate fashion at the abutments. This may require modification of approach embankments and abutments. Raising the superstructure will also affect the existing substructure. For example, the horizontal forces at raised bearings may cause more severe load effects in the substructure once the structure is raised. The materials used to raise the superstructure will also increase the dead load that the substructure must carry. It should be verified that existing substructures will not be adversely affected by these changes.

7.2.8 Connecting Beam Webs at Pier Bents

This retrofit measure consist of using restraint cables to connect the ends of beams in adjacent spans. Holes would be drilled in the end blocks of precast beams as well as through the end diaphragms, and a cable would be looped through and spliced. Significant vertical displacement between the two ends would be required to engage the restraining cable.

Consideration should be given to the potentially large impact forces arising from the large unrestrained vertical displacements. It may be desirable to place neoprene or other cushioning materials on the beam ends to prevent damage to the concrete when the two beam ends contact each other. Additionally, it may be advantageous to size the vertical bearings to limit damage to the beams and pier bents.

7.2.9 Connecting Diaphragms at Pier Bents

Provision of vertical shear continuity among spans can also be accomplished by connecting diaphragms across expansion joints at pier bents. In this retrofit concept, a steel pipe anchored to one diaphragm and free to slide in a hole in the other would provide the linkage. The pipe would behave as a cantilever beam, carrying the load in bending. Consideration of clearances in this retrofit concept will be important, as the annular space around the pipe in the expansion diaphragm must be sufficient to allow for the differential rotations of the two beam ends. Even with these considerations, this measure allows for much smaller differential vertical displacements before engaging than the cable restrainers attached to beam ends. The collateral damage should therefore be reduced.

7.2.10 Simple Spans Made Continuous

It may be possible, in some circumstances, to increase the resistance of a bridge to wave loads and traffic loads by providing continuity at the piers. This procedure, which is often used in new construction strictly to reduce the stresses due to live load, has been extensively treated elsewhere. For the purposes of this manual, the change from simple spans to continuous spans can increase the resistance of the bridge to wave/surge loads. Most of the published treatments of this procedure are concerned with the creep and shrinkage induced loads, which in new construction can be significant. It may be possible where this is used as a retrofit to neglect these effects as most of the creep and shrinkage strains have already occurred, and the new creep due to changes in stress (from prestress, etc.) should be small.

If a connection is provided to the substructure, positive moments may develop over the piers from buoyancy and wave loads. These may require reinforcing steel in both the top and bottoms of the connection between the beams. In addition to issues related to live load continuity, the connection between spans must also be capable of transmitting wave loads if live load continuity is to be used as a wave loading retrofit. The accomplishment of both objectives may not always be possible or practical.

7.2.11 Connection of Superstructure to Substructure

There are a myriad of measures that can be employed to effect a connection between the superstructure and the substructure. Some provide both vertical and lateral restraint, while others supply on vertical or lateral. As for all measures, a careful evaluation of the tolerances, the displacements required to engage the restraint, and the potential ductility of the restraint should be made to guard against a progressive failure of the restraint system at a net load level substantially lower than the design value.

7.2.11.1 Earwalls

Similar in concept to external shear keys often used in seismic design, the earwalls are intended to provide a physical restraint to the lateral movement of the superstructure. For a given direction of loading, all transverse loads at a pier will be carried by a single earwall. This eliminates the uncertainty in distributing the transverse load among several resistant elements. However, the forces to be resisted by this single element can become very large, and in the worst cases it will be unmanageable.

A steel or concrete extension is attached to the end of the bent cap and extends upward to prevent the external beam from displacing horizontally. Ideally there will be solid end diaphragms which will serve to distribute the load amongst all the girders. Likely a post-tensioning system will be required to attach the earwall to the bent cap.

7.2.11.2 Shear Blocks

A large number of variations are possible within the general shear block concept. All of them consist primarily of a concrete or steel addition to the pile bent cap between beams to prevent transverse displacement. Vertical restraint can also be accomplished by casting the shear block above and around the bottom flange of precast beam or adding steel brackets to the top of the shear block to engage the bottom flange.

The existing end diaphragms may have to be modified to provide clearance for shear block installation. In many cases this will dictate the shear block size.

7.2.11.3 Cable Restraints

Wire rope or other type of cables can be used to provide both vertical and lateral restraint. These cables can be looped around the bent cap and threaded through either the end blocks of the precast beams, holes drilled in the bottom flanges of steel beams, or through the end diaphragms of either steel or concrete beams. For additional lateral capacity, the cables could utilize the pile to bent cap intersection as a reaction point.

7.2.11.4 Cable Restraints to Piles

The bent cap may not have sufficient capacity to resist the loads applied by the cable restrainers, especially due to the vertical uplift forces. One measure that circumvents this issue is to tie the beams or diaphragms directly to the piles, bypassing the bent cap. Holes would be drilled in the piles to pass the cable loops through. Weakening of the piles by the presence of the holes, as well as the ability of the piles to carry the uplift and lateral loads should be considered.

7.2.12 Auxiliary Foundations

New foundation elements may be attached to the existing substructure through the bent cap to augment the load capacity of the existing foundation. When the superstructure is being restrained by both existing and auxiliary foundations, the amount of movement required to engage each element should be investigated to ensure that the two foundations will be engaged simultaneously as desired. Similarly, when horizontal forces are to be carried by the existing superstructure and vertical forces by the auxiliary foundation it must be confirmed that the connections provided and the relative stiffness of the foundations will permit this.

Auxiliary foundations may use elements such as spin-fin piles, micropiles, drilled shafts and prestressed soil anchors. Brief descriptions of these elements are given below:

- Spin-fin piles employ fins welded to pipe piles at an angle. The piles screw into the ground during driving. Spin-fin piles have greatly increased pullout capacity, especially under repeated loading.
- Micropiles are small diameter (typically less than 12 inch) piles that can be installed in almost any type of ground, and are particularly effective in restricted access or low headroom situations. Micropiles may provide substantial uplift capacity in comparison to driven piles or drilled shafts by virtue of their installation in conjunction with pressure grouting.

- Drilled shafts are reinforced concrete columns typically constructed using tremie placement methods within a cased hole ranging in size from 3’ to 12’ in diameter.
- Soil anchors offer an economical solution to temporary or permanent stability or support problems. They achieve relatively high uplift resistance through pressure grouting. Designed to resist uplift forces for this application, prestressed soil anchors can be designed to resist loads in excess of 100 tons depending on the size of anchor and type of soil in which the element is bonded.

7.2.13 Ground Improvement

Ground improvement may be used to improve both the geotechnical uplift and lateral resistance of bridge foundations. Depending on the foundation soil type(s), bridge structure constraints and environmental controls, ground improvement options may include vibro-compaction, vibro-replacement, and deep soil mixing and jet grouting.

- Vibro-compaction is used to densify clean, cohesionless soils using a vibrator and accompanying water jetting to increase the relative density of the soil to 70 percent to 85 percent.
- Vibro-replacement extends the range of soils that can be improved by vibratory techniques to include cohesive, mixed and layered soils. Densification and/or reinforcement of the soil with compacted granular columns is accomplished by either top- or bottom-feed methods.
- Deep soil mixing and jet grouting is a versatile soil replacement technique used to create in situ cemented inclusions of soilcrete, and is effective across a wide range of soil types, including silts and some clays.

The suitability of a ground improvement alternative will depend on the type and variability of soils into which the foundations are embedded. Ground improvement treatment will likely be limited to the perimeter of the foundations due to access issues.

There are limits to the benefits of these measures. The increase in geotechnical resistance, especially for uplift loading, may be limited:

- full treatment of the ground around the foundations may be difficult
- the increase in strength may not be sufficient even with full treatment
- the structural resistance of the foundation may not be adequate to resist wave force loads even if the geotechnical resistance can be increased sufficiently.

Jet grouting may have environmental constraints because it results in mixtures of cement, soil, and water being purged as grouting proceeds, a condition which may pose an environmental concern at some sites.

7.2.14 Enhance Structural Capacity of Substructure

This method may be used when the existing substructure has inadequate capacity to resist the surge/wave loads transferred from the superstructure. General strengthening methods include:

- Increasing the shear or moment capacity of pier caps and piles/columns using FRP sheets
- Tying the pier cap to the columns/piles to maintain the pier cap to column/pile connection or tying the superstructure directly to the piles/columns to avoid load transmission through the pier cap.

Existing bridge bents/piers may also be susceptible to damage due to lateral loading. As discussed earlier, the lateral capacity of a bent may be governed by the bending, shear, or tension capacity of the piles/columns or pier cap. It may be possible to increase the resistance of a bent/pier by increasing the strength of its individual members. However, strengthening piles at or below the water line or mud line may be impractical with respect to constructability.

7.2.15 Structural Fuses

Structural fuses must have a predictable failure load. Fuses made from significantly over-strength material will not break at the desired load. Thus materials used in the construction of structural fuses must have both a required minimum and maximum strength. Often, widely available components such as bolts and turnbuckles will have only a specified minimum strength or a maximum strength that is too much greater than the minimum strength. It will likely be necessary to seek specialty components or to have components custom fabricated from a highly controlled material.
To maintain a predictable failure load, the fuses must also resist deterioration which may reduce their breaking load. Only materials and coatings that are capable of withstanding extended periods in a marine environment should be considered for coastal storm retrofitting of bridges.

In addition to having predictable failure loads, fuses must also be loaded predictably. Connections which utilize several structural fuses in parallel may not provide predictable loading. If it is desired to add the resistance of each fuse to obtain a total resistance, it must be ensured that simultaneous engagement of the fuses will occur. If simultaneous engagement does not occur, one fuse at a time may be loaded and failed, and the desired total resistance will be significantly less than the sum of individual components. Thus it may be beneficial to use one or two large capacity fuses instead of several fuses with a smaller capacity. Impact loads in loose connections may also lead to fuses that are not loaded in a predictable fashion. Fused connections should generally be “snug” so that impact is avoided, however, the connection should still avoid transmitting loads under typical operating conditions.

7.2.16 Bearing Modification

It is likely that the reactions delivered to beams by bearings during surge or wave events will differ considerably from those under typical service conditions. It may be possible to limit the damage to beam seats and pier caps by providing oversized (thickness and area) neoprene bearing pads. Increased thickness may protect the beams and pier cap by cushioning the impact if a beam is dropped/slammed during a surge or wave event. Increased bearing pad area may protect the beams and pier cap in the event that bearing area is lost due to beam shifting or pier cap damage. Oversized neoprene pads should be fastened to either the beam or pier cap so that they are not dislodged if a beam is lifted or shifted. If the possibility exists for significant superstructure displacement, fastening bearing pads to the beams would be preferred. By fastening the bearing pads, spalling type damage due to concrete on concrete bearing may be mitigated. Oversized pads will also provide an increased bearing area which may protect the beam and pier cap from increased reaction forces sustained during surge/wave events.

It should be noted that oversized bearing pads must still satisfy applicable design criteria for typical operating conditions. When thicker bearing pads are considered as a retrofit alternative, issues such as bearing pad stability and increased force effects on anchor bolts should be investigated.

CORED DECK VENTS

General Retrofit Method:
Prevent entrapment of air under superstructure.

General Retrofit Principal:
Provide a path for air to escape from under superstructure.

Specific Retrofit Method:
For each space where air could be trapped, core a hole in the deck and line it with a PVC (or similar) pipe secured with epoxy.

Specific Retrofit Concept:
Permit air to escape through holes in deck.

Notes:
Where possible place vents in the center of the lanes to minimize the occurrence of vehicle wheels passing over the vents.

Provide covers over vents to improve ride quality and reduce impact.

If deck is sloped longitudinally or transversally, place vents near the highest point of each air cavity to minimize the volume of remaining trapped air.

If possible, place vents to clear reinforcement.

Pros:
Application simplicity.

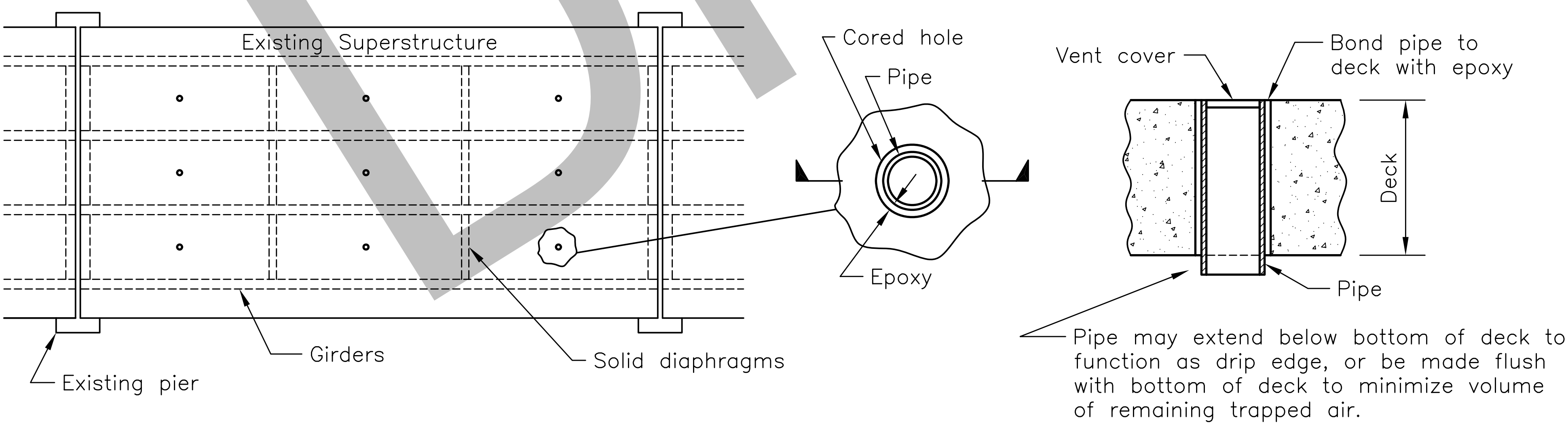
Cons:
May not vent air fast enough.

Solid vent covers will require removal before storm events while screen type covers may restrict air flow.

Manual removal of vent covers may require lane closures during critical evacuation periods.

Blow—off type vent covers and water spouting through vents may cause hazardous conditions for motorists and pedestrians during wave events, appropriate precautions should be taken to avoid this.

Analysis Issues:
Determination of the required hole diameter may require knowledge of both pneumatics and wave mechanics.



DIAPHRAGM VENTS

General Retrofit Method:
Prevent entrapment of air under superstructure.

General Retrofit Principal:
Provide a path for air to escape from under superstructure.

Specific Retrofit Method:
Core holes in each solid diaphragm.

Specific Retrofit Concept:
Permit air to escape through holes in diaphragm.

Notes:
Place vents as close to deck as possible to minimize remaining volume of trapped air.

If possible place vents to clear reinforcement.

Coat inside of cored hole with epoxy or other protective coating to protect cut reinforcement or reinforcement with reduced cover, if desired.

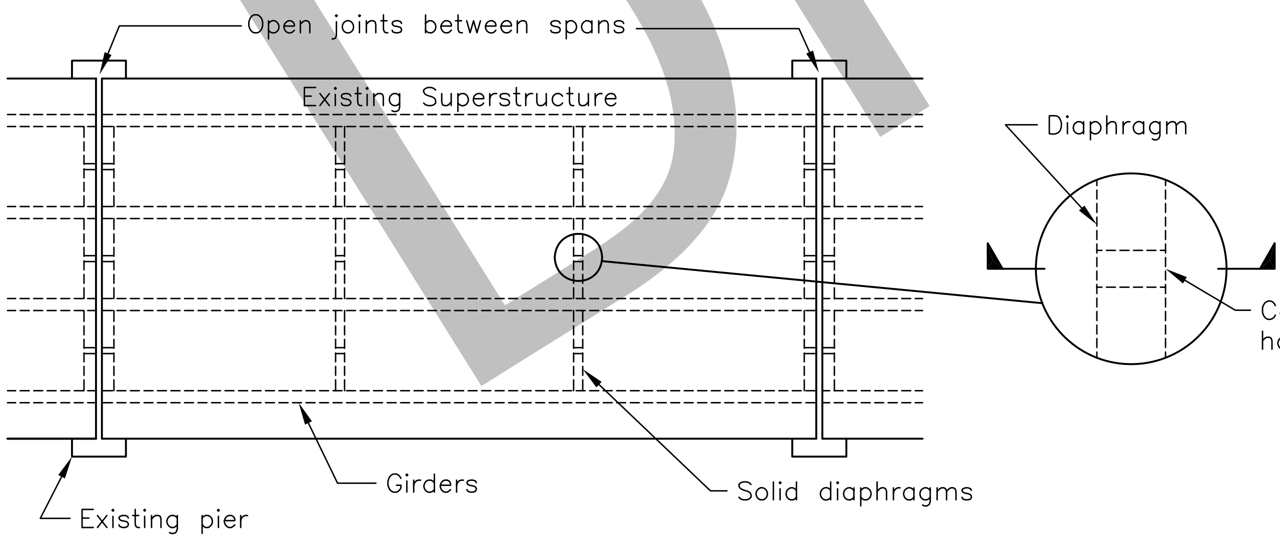
Pros:
Application simplicity.

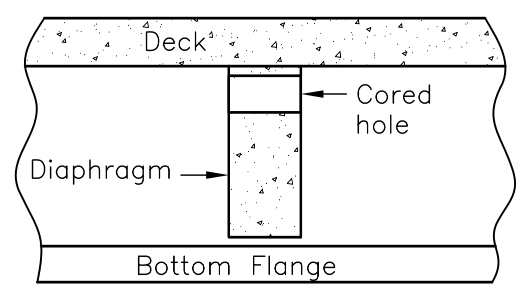
No traffic issues—does not require holes through deck.

Cons:
May not vent air fast enough.

A path for air to escape must be present at ends of span.

Analysis Issues:
Determination of the required hole diameter may require knowledge of both pneumatics and wave mechanics.





REPLACEMENT OF SOLID DIAPHRAGMS

General Retrofit Method:
Prevent entrapment of air under superstructure.

General Retrofit Principal:
Provide a path for air to escape from under superstructure.

Specific Retrofit Method:
Replace solid diaphragms with steel frame diaphragms.

Specific Retrofit Concept:
Permit air to escape by removing solid diaphragms.

Notes:
Ensure reinforcement and prestressing strands are not damaged if beams are drilled.

Provisions should be made to permit future jacking of the superstructure if possible.

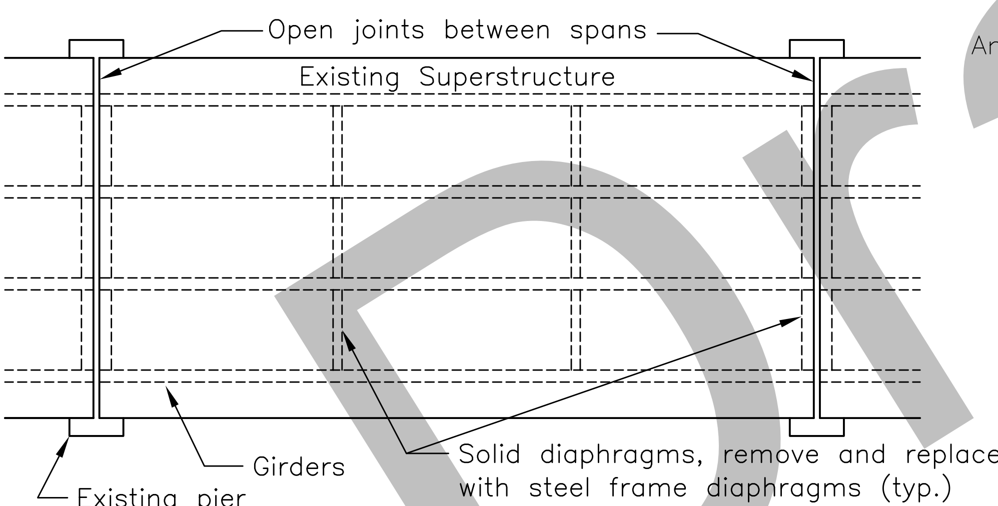
Pros:
Steel frame diaphragms will not obstruct air flow.

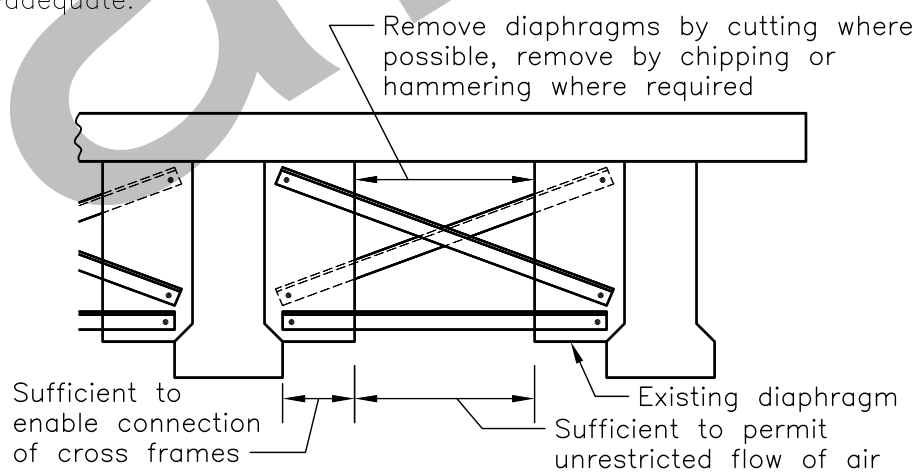
Cons:
Requires removal of solid diaphragms.

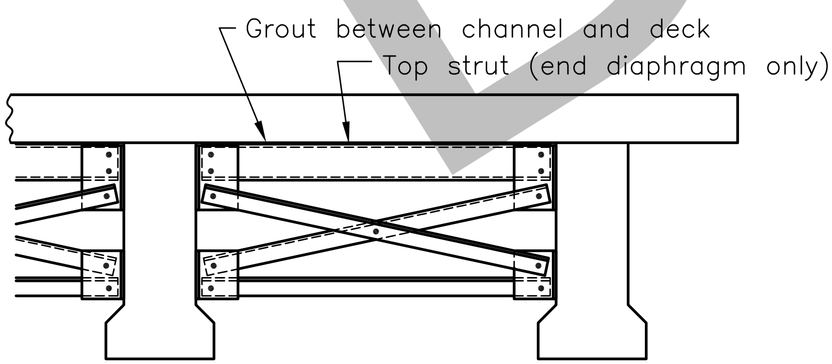
May require drilling of beams.

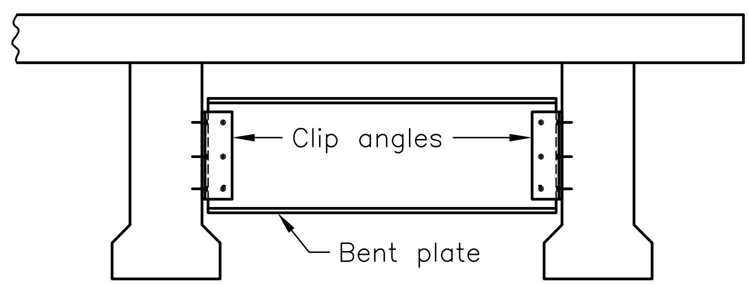
A path for air to escape must be present at ends of span.

Analysis Issues:
Connections between the steel framing and girder must be adequate.









BREAK-AWAY BARRIER

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Reduce projected area of waveward side of bridge.

Specific Retrofit Method:
Install directional breakaway barriers.

Specific Retrofit Concept:
When wave forces on the bridge barrier reach a specified level, the barrier will break away, thus reducing the area of bridge subjected to wave forces.

Notes:
Barriers must be crash worthy to prevent vehicles from breaking through the barriers and leaving the bridge.

Barriers should lean-over onto bridge deck when subjected to wave forces of a predetermined magnitude.

Breakaway barriers are only needed on the waveward side of the bridge.

Pros:
Precast barriers may facilitate quick repair after a wave event.

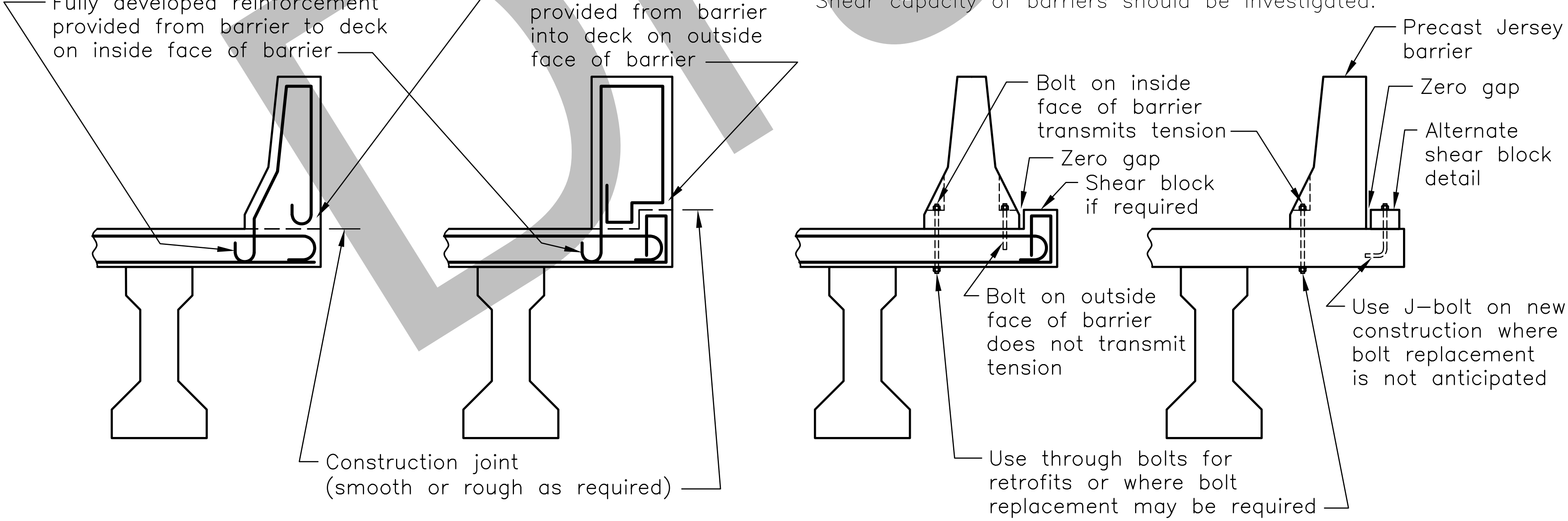
Cons:
Cast-in-place barriers may be difficult to repair after a wave event.

Analysis Issues:
Barriers must be crashworthy.

Breakaway load of barriers must be predictable.

Plans for barrier repair should be made during design.

Shear capacity of barriers should be investigated.



SLAB SPANS/ADJACENT BOX BEAMS

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Alter the geometry of the bridge cross section.

Specific Retrofit Method:
Replace I–girder type spans with slab spans or adjacent box beams.

Specific Retrofit Concept:
Reduce the area of vertical surfaces exposed to horizontal pressures due to surge/waves and raise the bottom of girder elevations.

Notes:
May be suitable for low spans where bridge meets grade or for replacement of spans already lost to a coastal storm.

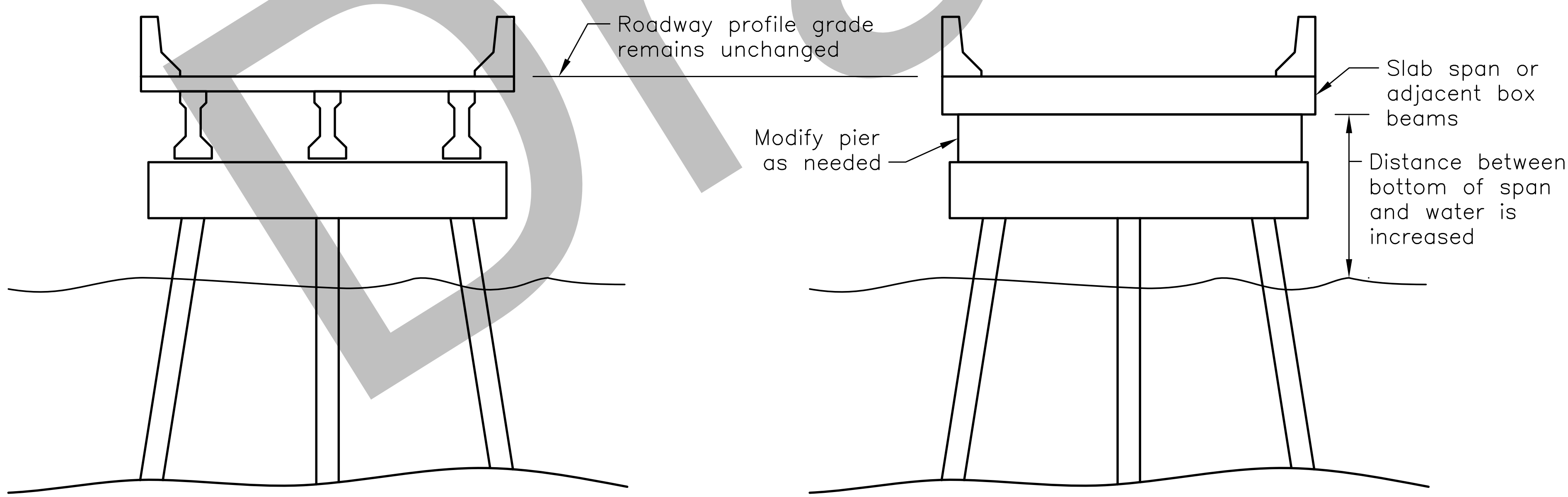
May be suitable for new designs.

Pros:
May significantly reduce wave/surge loads on bridge spans.

Cons:
Requires complete replacement of spans.

Slab spans and adjacent box beams may not be the most effecient span types for highway loads.

Analysis Issues:
Methods of wave/surge load estimation based on I–girder type cross sections may not be valid for slab and adjacent box beam spans.



EXISTING STRUCTURE

RETROFITTED STRUCTURE

ARTIFICIAL REEF

General Retrofit Method:
Reduce wave forces on bridge.

General Retrofit Principal:
Reduce wave height at bridge site.

Specific Retrofit Method:
Install an artificial reef on the seaward side of the bridge to change the bathymetry.

Specific Retrofit Concept:
Use an artificial reef to limit the height of waves impacting the bridge.

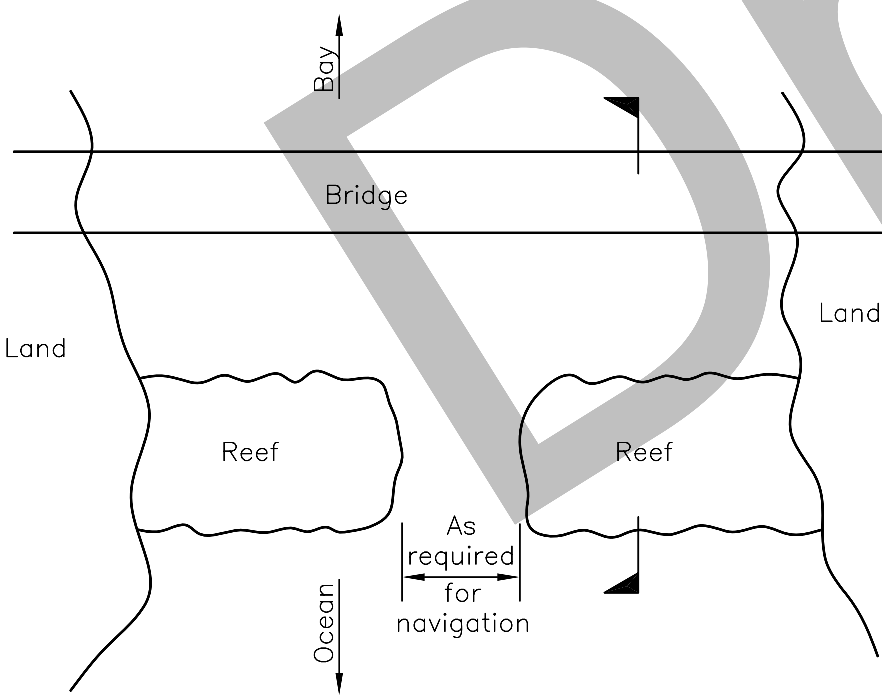
Notes:
May be appropriate for protecting portions of the bridge such as the abutments and/or the lowest elevation spans.

Pros:
Reef may provide recreational benefits to anglers and divers by providing habitat for fish.

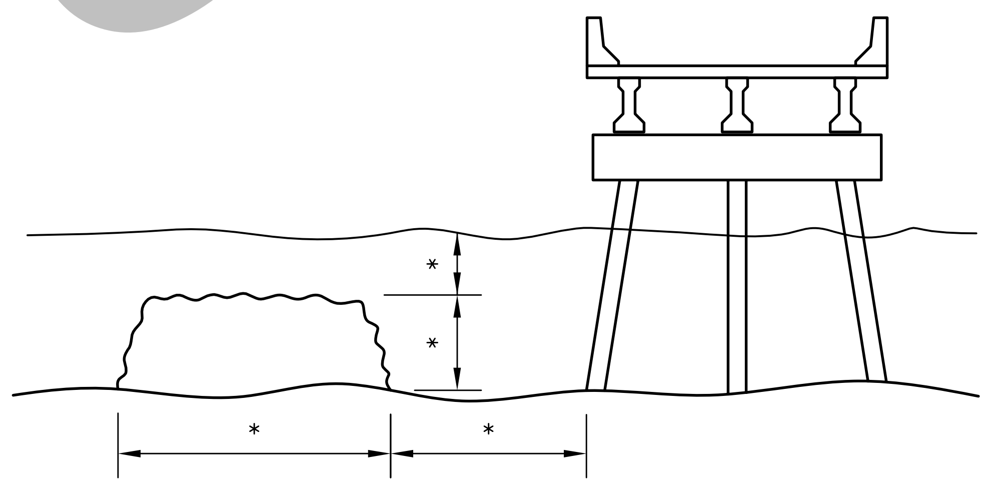
Cons:
Possibility exist for environmental, navigational, and scour issues.

Large amount of material required to construct reef.

Analysis Issues:
A site specific investigation by a qualified coastal engineer will be required to determine the effectiveness of this option and the required location and dimensions of the reef.



PLAN



* Dimensions to be determined based on site specific information and thorough investigation by a coastal engineer.

ELEVATION

BEAM TO BEAM CABLE RESTRAINTS

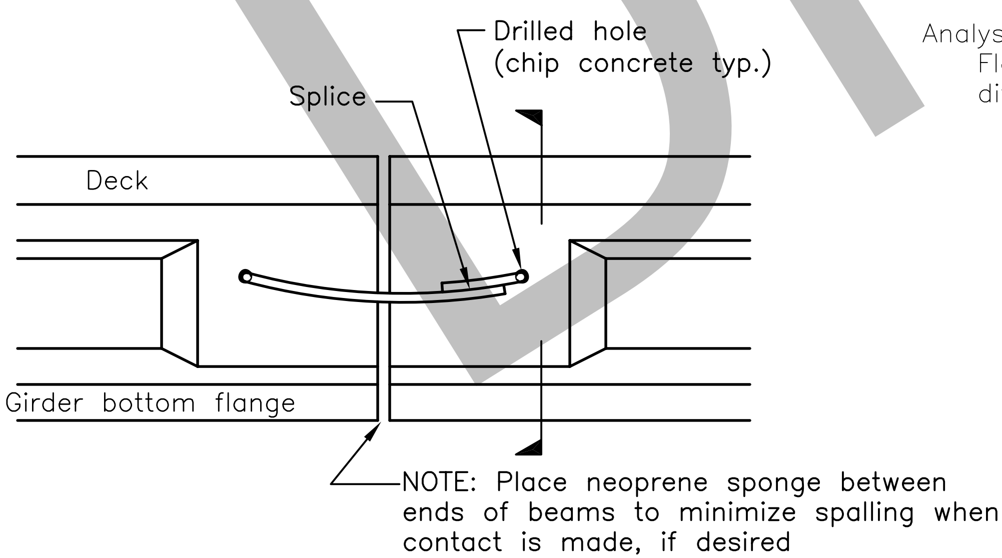
General Retrofit Method:
Tie spans together.

General Retrofit Principle:
When one span is experiencing maximum loading, adjacent spans will/may experience loads with a smaller magnitude. Tieing spans together will permit a span experiencing maximum load to engage the dead load resistance of an adjacent span.

Specific Retrofit Method:
Use steel cables to connect beam webs.

Specific Retrofit Concept:
Transfer wave loads from web of one beam to web of the corresponding beam in the adjacent span through cables while not transferring loads under typical operating conditions.

Cables Transmit:
- Vertical force as one beam lifts.
- Horizontal force as one span shifts laterally.
- Longitudinal tension forces.



Notes:
Longitudinal compression will be transferred when beam ends contact.

Where cables wrap around concrete corners, round corners by chipping to prevent kinking of the cable.

Locate holes as far from end of beam as practicable to reduce drilling in high stress areas. Longer cables will also be able to dissipate more energy.

Locate holes to clear reinforcing and prestressing.

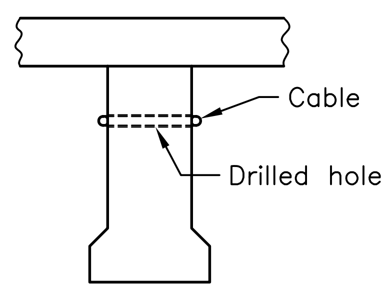
Where cables wrap around concrete corners, chip the corners to prevent kinking at the cables.

Pros:
Capable of transmitting lateral, longitudinal, and vertical forces.

Cons:
Requires drilling in areas of high stress and congested reinforcement.

Beams without enlarged end cross section will be more sensitive to drilling

Analysis Issues:
Flexible nature of connections will allow possibly significant differential movement before the cables are fully engaged.



STEEL DIAPHRAGM PIPES

General Retrofit Method:
Tie spans together.

General Retrofit Principle:
When one span is experiencing maximum loading, adjacent spans will/may experience loads with a smaller magnitude. Tying spans together will permit a span experiencing maximum load to engage the dead load resistance of an adjacent span.

Specific Retrofit Method:
Core holes in end diaphragms and insert concrete filled steel pipe (Detail 1). Provide pipe capable of transmitting longitudinal loads in new or existing diaphragms (Detail 2).

Specific Retrofit Concept:
Transfer wave loads from one span to another through the full depth end diaphragms while <u>not</u> transferring loads under typical operating conditions.

Pipes Transmit:
- Vertical shear as one span lifts.
- Horizontal shear as one span shifts laterally.
- Longitudinal load as spans shift longitudinally (Detail 2).

Notes:
Ensure length of pipe is less than length of cored hole so that the pipe will never be in contact with the cover plate when joint is closed (Detail 1).

The diameter of the cored hole and pipe should be selected so that the pipe is not engaged due to live load rotation of the girders.

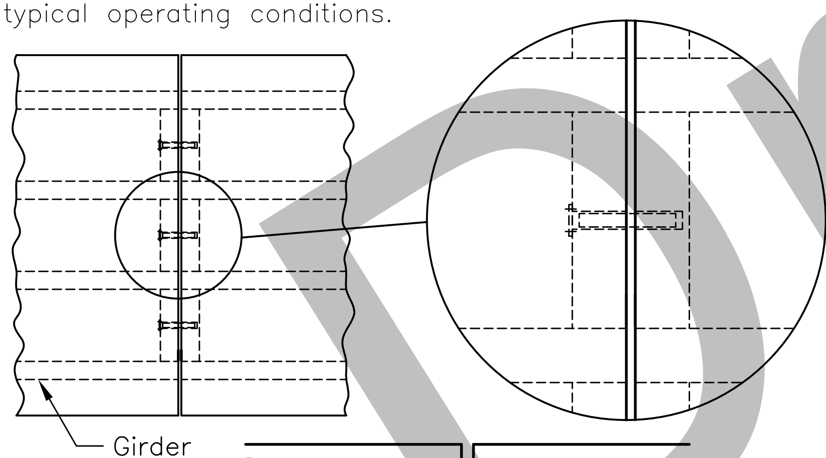
Multiple connections per diaphragm may be used if needed.

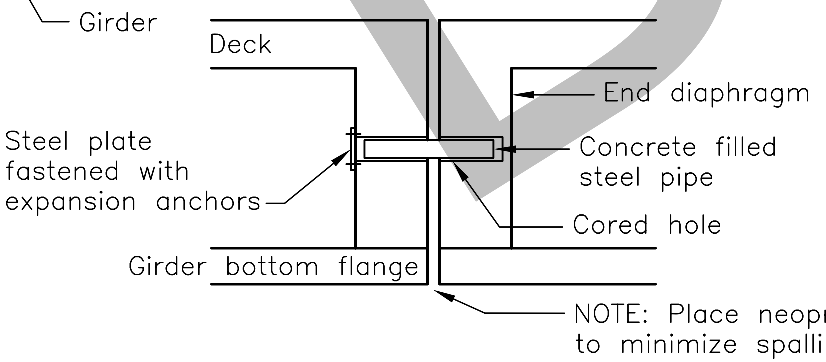
Pros:
Does not require drilling of beams.

Transmits lateral and vertical loads as well as longitudinal loads if desired.

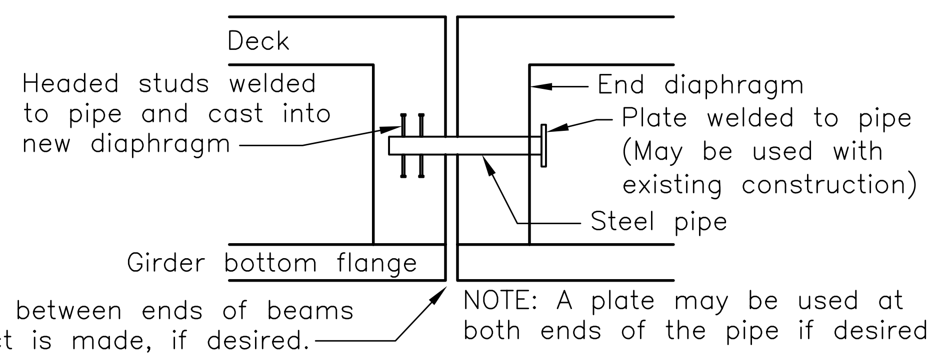
Cons:
Connection of adjacent spans could result in displaced spans dragging adjacent spans off of their supports.

Analysis Issues:
Beam to diaphragm connections must be adequate to transmit loads.





DETAIL 1



DETAIL 2

EARWALLS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Anchor earwall of steel, concrete or other suitable construction to end of existing pier cap.

Specific Retrofit Concept:
Transfer lateral wave loads from fascia beams into pier cap through earwall.

Notes:
Adequate bearing surface between earwall and fascia beam should be provided if possible.

Pros:
Provides a method of providing lateral restraint when the configuration of the beams, diaphragms, and/or pier cap are not conducive to other connection methods.

Does not require extensive work to be performed in the confined area between girders.

Earwall may be fabricated off site.

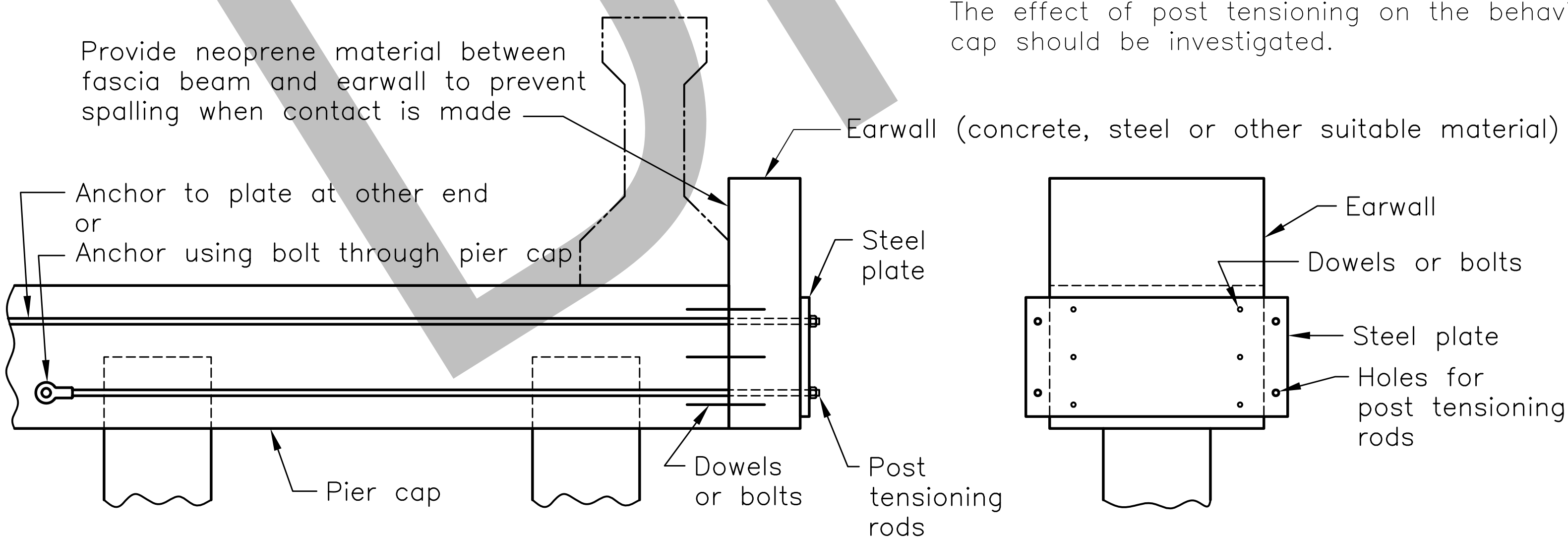
Post tensioning force may also be used as a strengthening measure for the pier cap.

Cons:
Provides only lateral restraint.

Analysis Issues:
Earwall must have a high shear capacity.

Fascia beam must be capable of transferring reaction force into the superstructure.

The effect of post tensioning on the behavior of the pier cap should be investigated.



LOW SHEAR BLOCKS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use concrete shear blocks anchored to existing pier cap.

Specific Retrofit Concept:
Concrete shear blocks will transfer lateral loads from the beams into the existing pier.

Notes:
To ensure concrete does not get under beam fill space around bearing pad with Styrofoam or other soft material during casting of new shear block.

Cast with neoprene sheet separating shear block from beam to facilitate possible future jacking of superstructure.

Pros:
High shear capacity may be obtained.

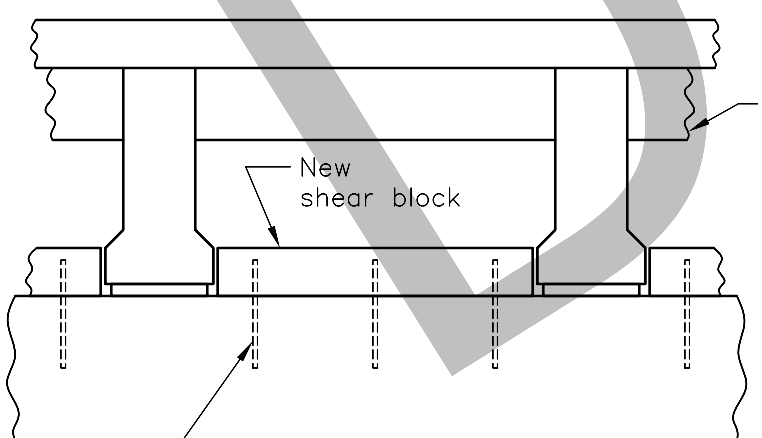
Use of low shear blocks in combination with other retrofit methods may be possible.

Cons:
Placement of concrete may be difficult in confined area.

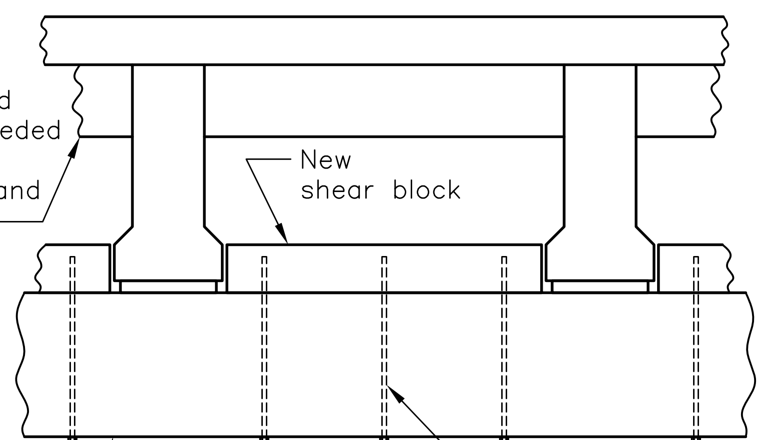
Provides only lateral restraint.

Analysis Issues:
Interface shear transfer between the new and old concrete should be carefully considered, roughening of the existing concrete may be desirable.

Stability of girders may be a concern if end diaphragms are not present.



Modify existing end diaphragms as needed to facilitate shear block installation and air venting



HIGH SHEAR BLOCKS WITH PROTRUSIONS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principal:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use concrete shear blocks anchored to existing pier cap.

Specific Retrofit Concept:
Concrete shear blocks will transfer lateral and uplift loads from the beams into the existing pier.

Notes:
To ensure concrete does not get under beam fill space around bearing pad with Styrofoam or other soft material during casting of new shear block.

To prevent restraint of the girder by the shear block due to girder live load rotation or thermal movement of the girder, provide a neoprene sponge of sufficient thickness between the girder and the shear block.

Pros:
High shear capacity may be obtained.

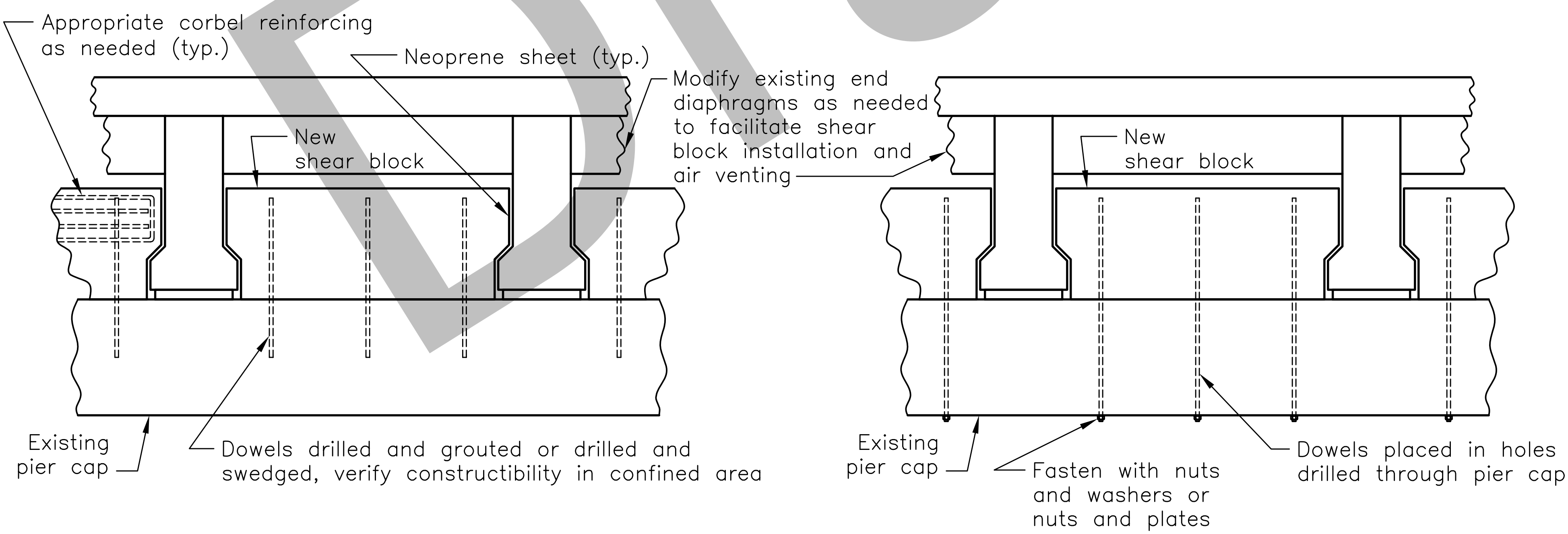
Provides lateral and vertical restraint.

Cons:
Placement of concrete may be difficult in confined area.

Does not allow subsoquent jacking of superstructure for maintenance activities.

Analysis Issues:
Interface shear transfer between the new and old concrete should be carefully considered, roughening of the existing concrete may be desirable.

The bottom flange of the girder must be capable of transferring the uplift reaction into the cross section.



HIGH SHEAR BLOCKS WITHOUT PROTRUSION

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principal:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use concrete shear blocks anchored to existing pier cap.

Specific Retrofit Concept:
Concrete shear blocks will prevent lateral movement of the beams while allowing the superstructure to rise under uplift forces.

Notes:
To ensure concrete does not get under beam fill space around bearing pad with Styrofoam or other soft material during casting of new shear block.

Use thick, oversized neoprene bearing pads to cushion beams as they fall back into place after an uplift event.

Securely fasten neoprene bearing pads to beam or pier cap to ensure that they stay in place during an uplift event.

Pros:
High shear capacity may be obtained.

Restrains superstructure laterally allowing it to move vertically a significant amount.

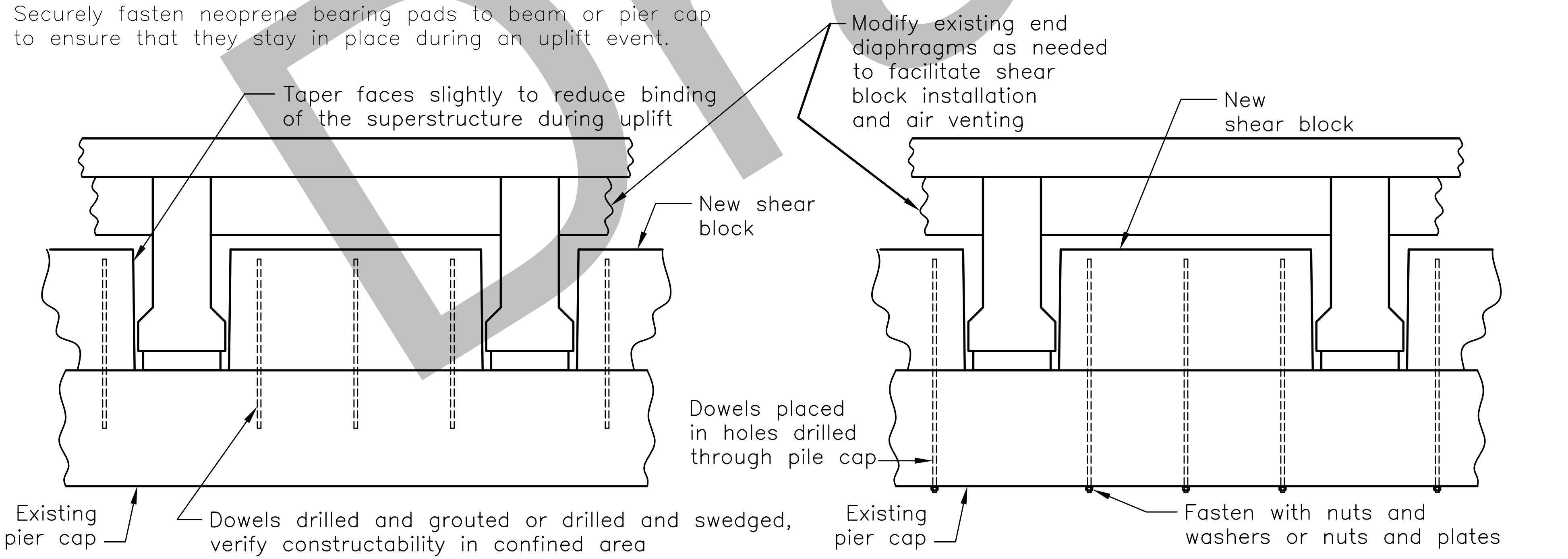
Does not transmit uplift loads to foundation.

Cons:
Placement of concrete may be difficult in confined area.

Superstructure may shift laterally if lifted above shear blocks.

Analysis Issues:
Interface shear transfer between the new and old concrete should be carefully considered, roughening of the existing concrete may be desirable.

The effects of dropping/slamming of the superstructure should be investigated.



SHEAR BLOCKS WITH STEEL HOLD DOWNS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use steel brackets anchored to existing pier cap.

Specific Retrofit Concept:
Steel brackets will transmit uplift loads from beams into the existing pier.

Notes:
Provide adequate clearance between the girder and bracket so that the bracket is not engaged due to live load rotation of the girder.

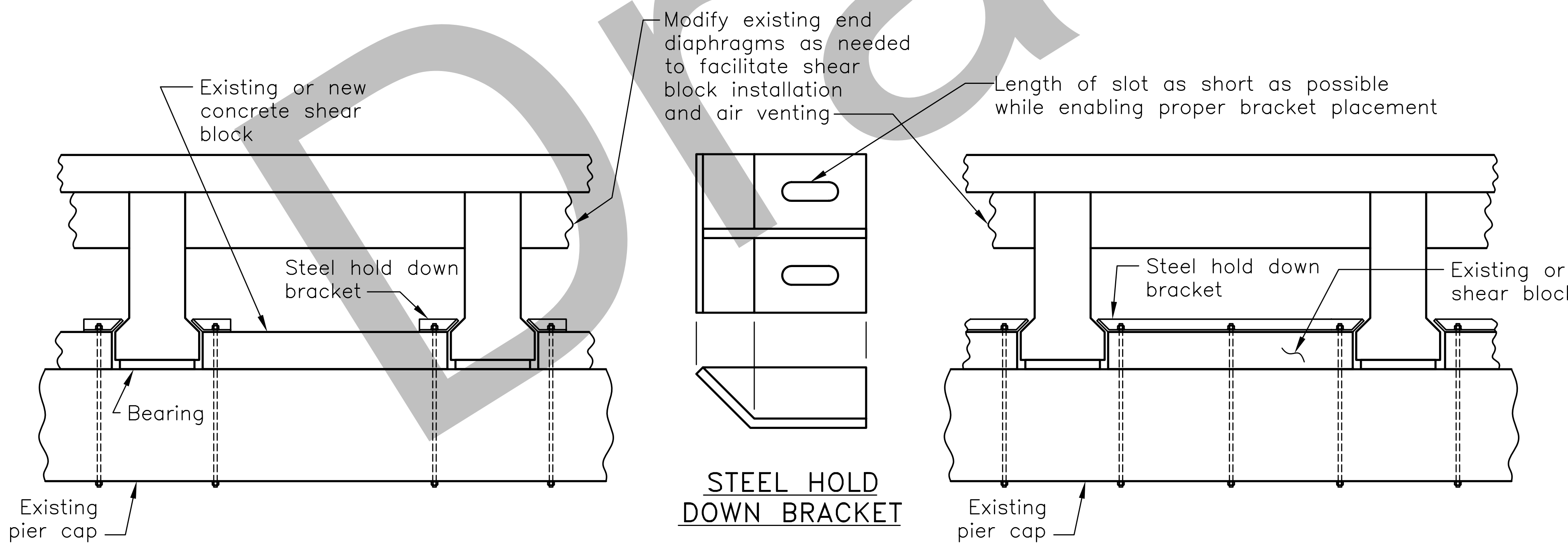
Pros:
May be used with new or existing concrete shear blocks.

Both lateral and uplift loads are resisted.

Cons:

Analysis Issues:
Stability of girders may be a concern if end diaphragms are not present.

The bottom flange of the girder must be capable of transferring uplift reaction into the cross section.



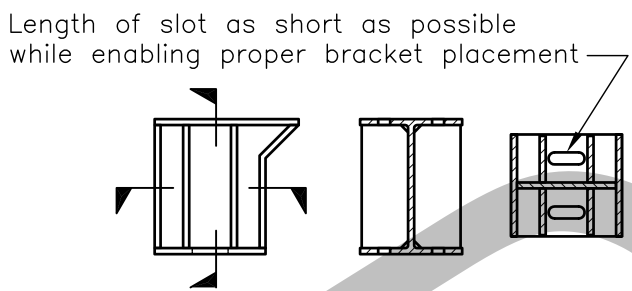
STEEL SHEAR BLOCK/HOLD DOWN BRACKETS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use steel brackets anchored to existing pier cap.

Specific Retrofit Concept:
Steel brackets will transmit lateral and uplift loads from beams into the existing pier.



SHEAR BLOCK AND HOLD DOWN BRACKET

Notes:
Provide adequate clearance between the bracket and girder so that the bracket is not engaged due to live load rotation of the girder.

Fabricate brackets starting with wide flange sections.

Pros:
Both lateral and uplift loads are resisted.

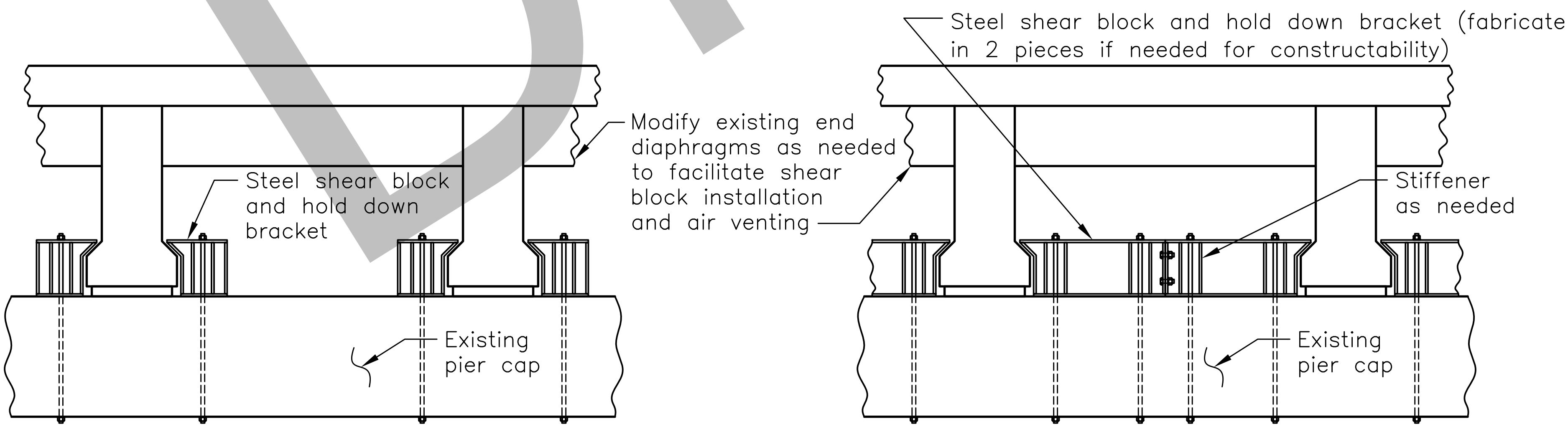
Does not require extensive field fabrication or concrete placement.

Brackets may be unbolted and temporarily removed for bearing maintenance.

Cons:
Geometry of the brackets may become complex for sloped pier caps, each bracket may require different dimensions. However, shims and/or beveled sole plates may be used to mitigate this problem.

Analysis Issues:
Stability of girders may be a concern if end diaphragms are not present.

The bottom flange of the girder must be capable of transferring uplift reaction into the cross section.



BEAM TO PIER CAP CABLE RESTRAINTS

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use cables or bars, which wrap around the pier caps to anchor girders.

Specific Retrofit Concept:
Cables or bars will transmit uplift wave forces to the piers.

Notes:
Where cables or bars wrap around existing concrete corners, round the corners by chipping to prevent kinking of the cable or bar.

Drill holes to clear reinforcing and prestressing.

Pros:

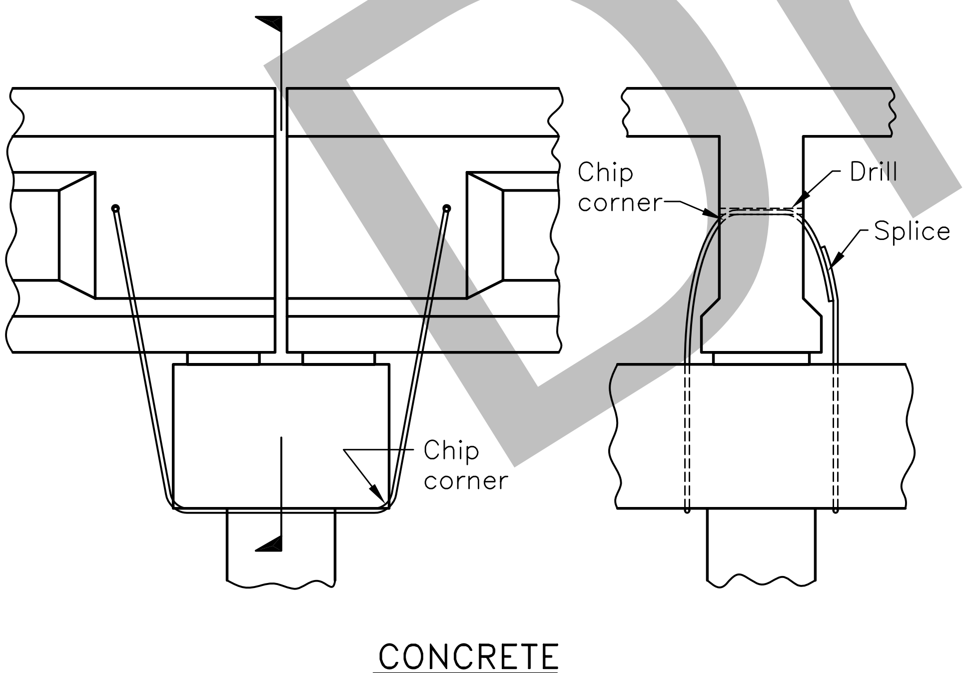
Requires minimal modification to pier cap beam.

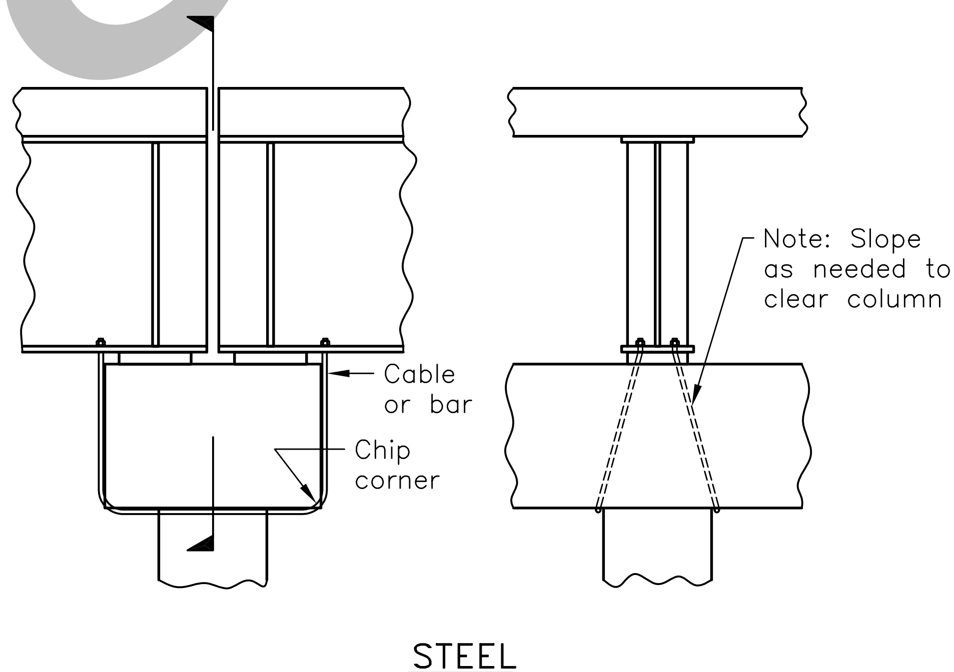
Under some circumstances it is possible that the connection may be designed to provide lateral as well as vertical restraint.

Cons:

Requires drilling of beams.

Analysis Issues:
Flanges of steel beams must be capable of transmitting the uplift reaction into the cross section.





DIAPHRAGM TO PIER CAP CABLE RESTRAINT

General Retrofit Method:
Anchor spans to existing pier.

General Retrofit Principle:
When existing piers have adequate capacity to resist wave forces, provide an adequate connection to transfer the force from the superstructure to the substructure.

Specific Retrofit Method:
Use cables, which wrap around the piers and go through holes or vents in the end diaphragms.

Specific Retrofit Concept:
Transfer uplift (and possibily lateral) wave loads from the end diaphragms to the pier through cables.

Notes:
Where cables wrap around concrete corners chamfer or chip the corners to prevent kinking of the cables.

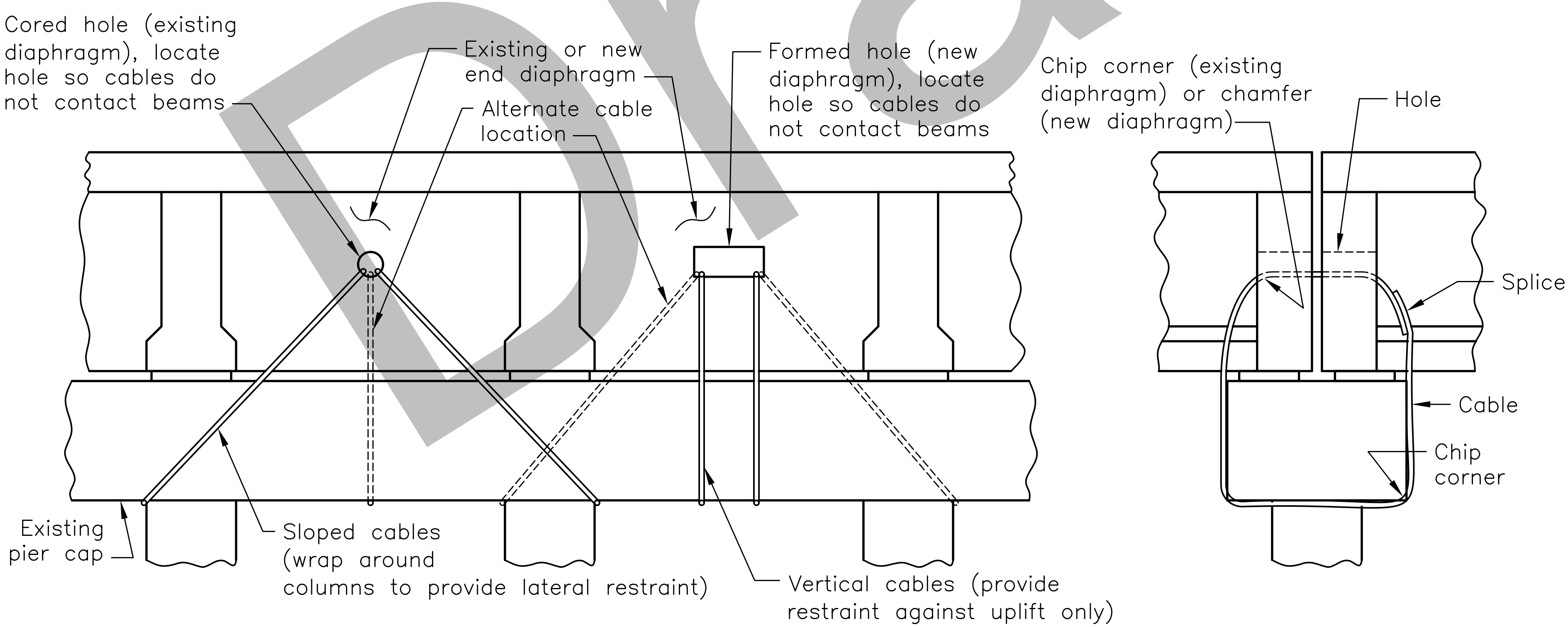
Pros:
May be designed to resist both uplift and lateral load.

May be used when face of end diaphragm is or is not flush with pier cap.

Holes in diaphragms may also function as air vents under some circumstances.

Cons:

Analysis Issues:
Beam to end diaphragm connections must be adaquate to transmit loads.



VIBRO–COMPACTION AND VIBRO–REPLACEMENT

General Retrofit Method:
Improve the soils surrounding the existing foundation elements.

General Retrofit Principal:
Where the existing substructure has inadequate capacity to resist wave forces, increase the capacity by improving the soils surrounding existing foundation elements.

Specific Retrofit Method:
Use vibro–compaction or vibro–replacement to improve soils surrounding existing foundation elements.

Specific Retrofit Concept:
The capacity of the existing substructure will be increased by ground improvement methods of vibro–compaction and vibro–replacement.

Notes:
Damage to the existing piles must be prevented. Care should be taken to prevent construction equipment from unnecessarily contacting existing piles, especially below grade where damage can not be inspected. The location of battered piles below grade should be considered.

The potential for piles to be subjected to bending induced by lateral pressures should be investigated.

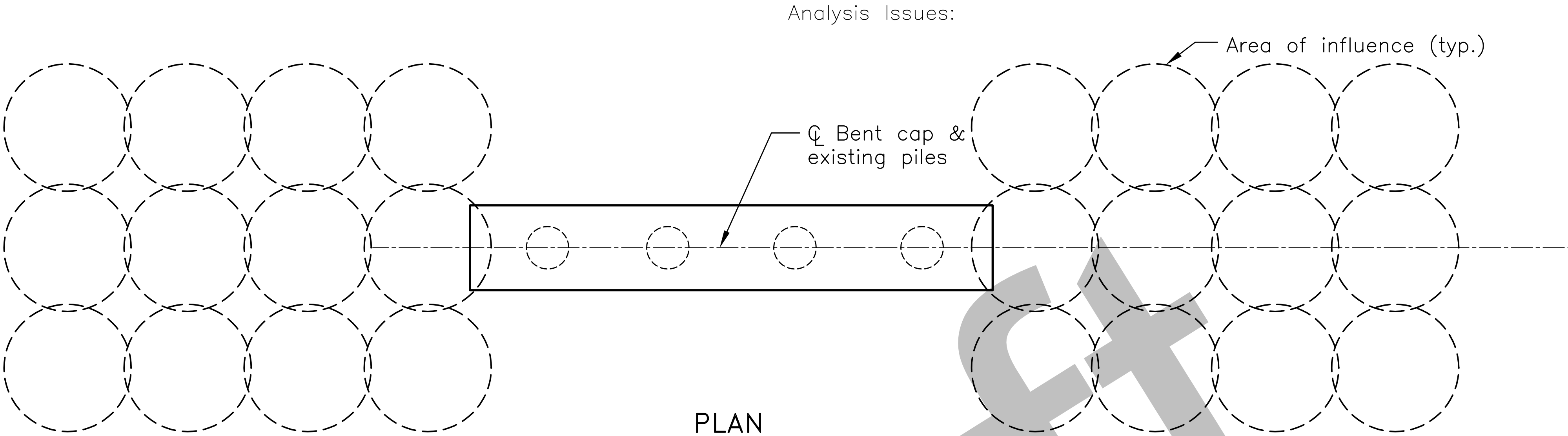
Pros:
Will not require lane closures.

Will not require new structural elements.

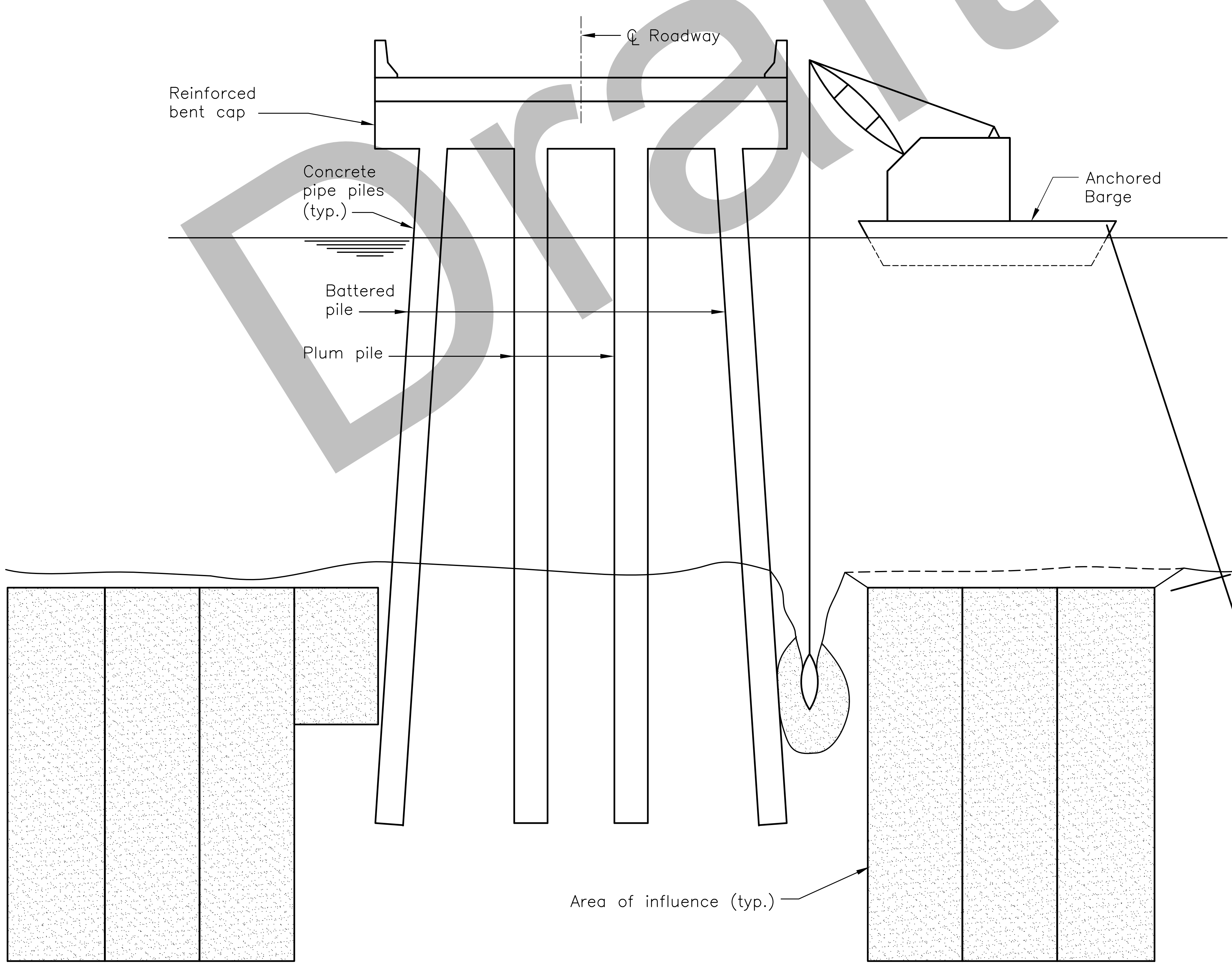
Cons:
The resulting increase in capacity may not be sufficient to resist wave loads.

Method is applicable only to clean cohesionless soils.

Analysis Issues:



PLAN



ELEVATION

PILE/COLUMN/PIER CAP STRENGTHENING USING STEEL SHELLS OR FRP

General Retrofit Method:
Strengthen existing substructure.

General Retrofit Principle:
When existing substructures have inadequate capacity to resist wave forces, strengthen them.

Specific Retrofit Method:
Increase strength using FRP wrap, FRP sheets, or steel shell encasement.

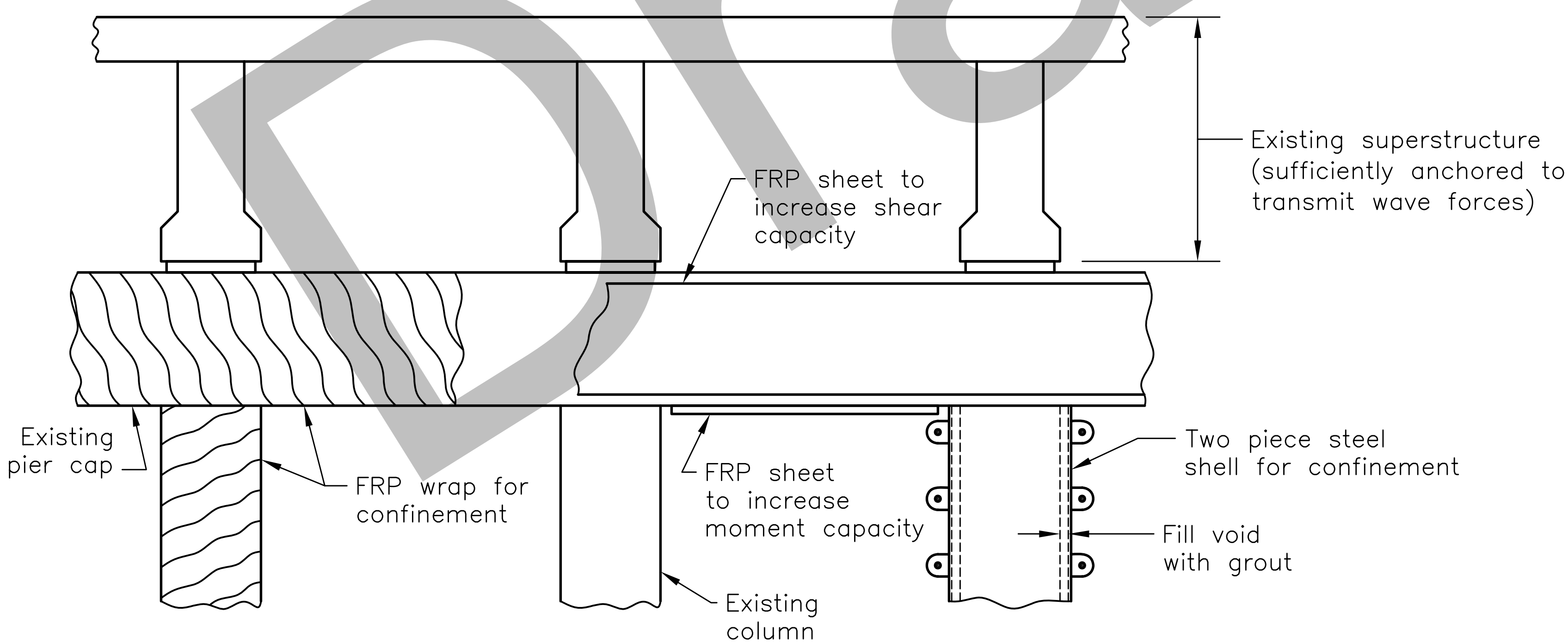
Specific Retrofit Concept:
Strengthen pier components to permit them to transmit wave forces from the superstructure into the foundation.

Notes:
The possibility of reinforcement corrosion due to entrapment of moisture by FRP or steel shells should be investigated.

Pros
Speed and simplicity of application.

Cons:

Analysis Issues:



PIER CAP TO COLUMN/PILE CABLE RESTRAINTS

General Retrofit Method:
Strengthen existing substructure.

General Retrofit Principle:
When existing substructures have inadequate capacity to resist wave forces, strengthen them.

Specific Retrofit Method:
Use cables, which wrap around pile cap and go through column/pile.

Specific Retrofit Concept:
Provide a connection capable of transmitting wave uplift forces from the pile cap into the columns/piles.

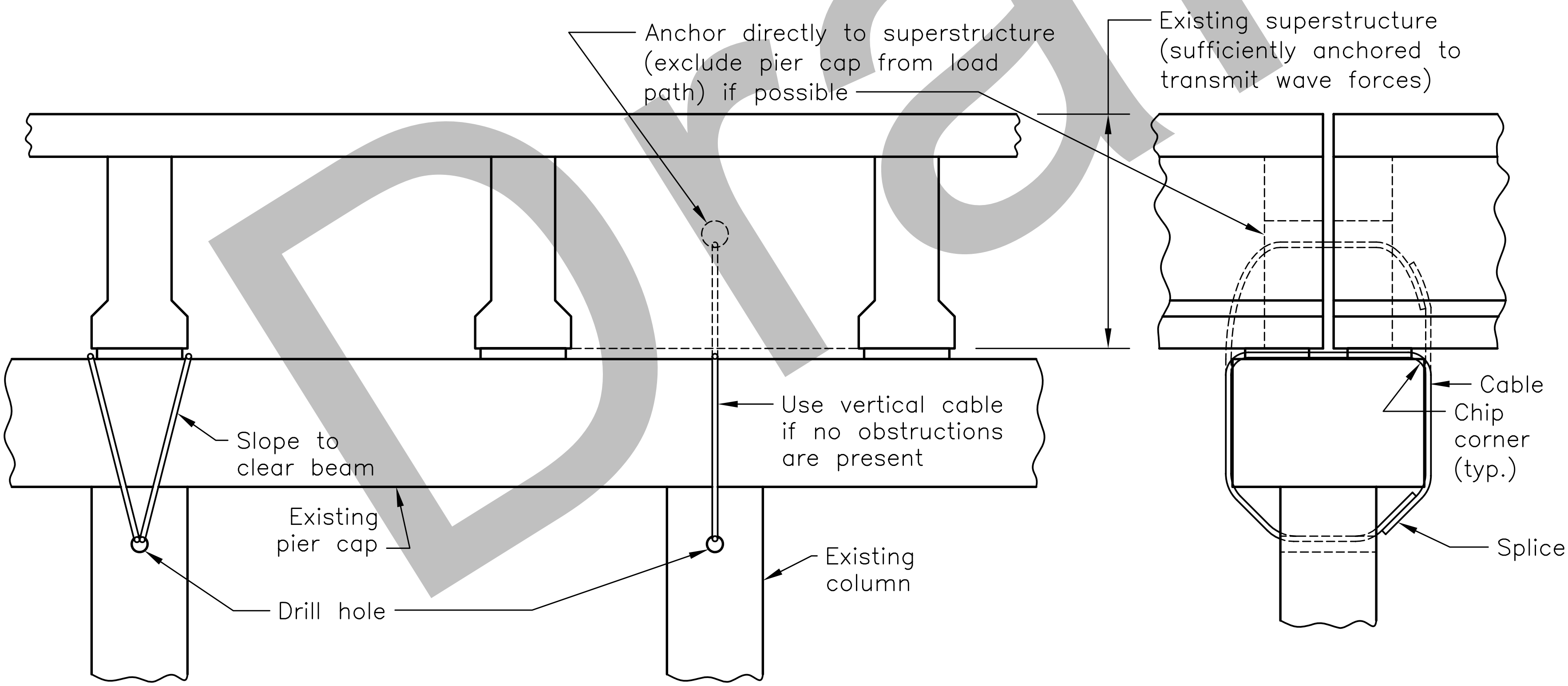
Notes:
When an insufficient connection for uplift is present between the pier cap and columns/piles means of load transfer must be provided.

In some cases retrofits concepts that anchor the superstructure to the pile cap may be modified to anchor the superstructure directly to the columns/piles.

Pros:

Cons:

Analysis Issues:



FUSED TURNBUCKLE BODY

General Retrofit Method:
Provide a fused connection.

General Retrofit Principle:
Provide a fused connection to mitigate damage to connected structural members.

Specific Retrofit Method:
Fuse cable connections.

Specific Retrofit Concept:
Use a specially fabricated turnbuckle to fuse a cable connection.

Notes:
Fabricate turnbuckle body using material with highly predictable failure load.

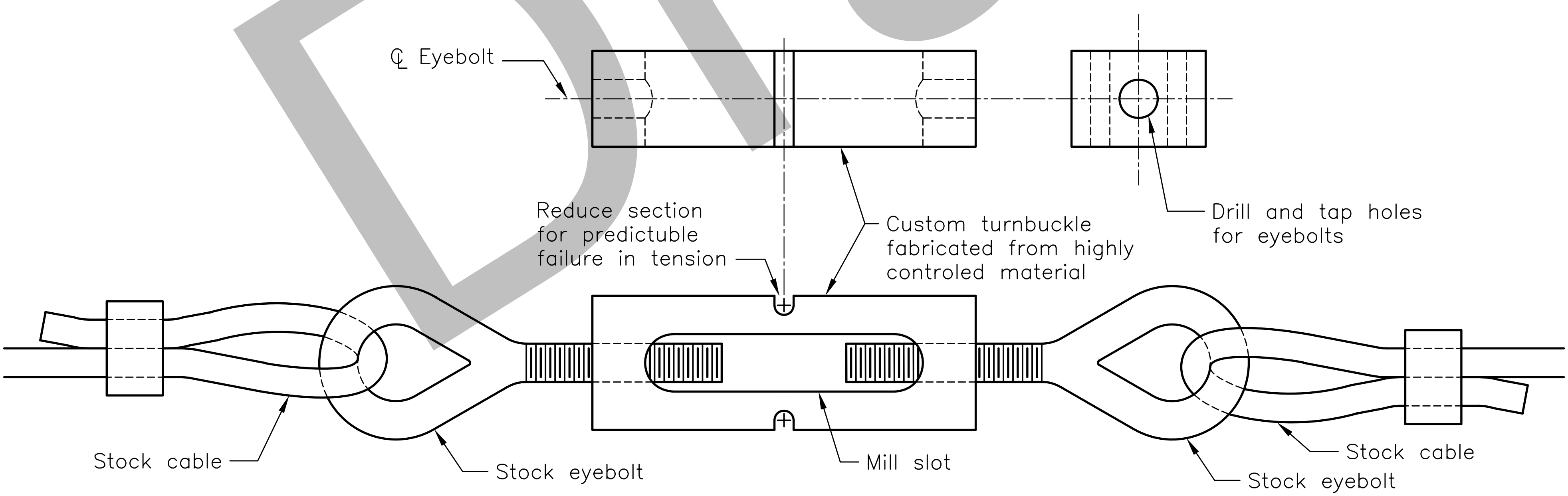
Provide dimensional tolerances to fabricator.

Ensure reduced section will produce the controlling load.

Pros:
Allows easy adjustment of cable lengths, while also functioning as a structural fuse.

Cons:
Requires custom fabrication.

Analysis Issues:
Determination of acceptable dimensional and material–strength tolerances.



FUSED TURNBUCKLE EYEBOLTS

General Retrofit Method:
Provide a fused connection.

General Retrofit Principle:
Provide a fused connection to mitigate damage to connected structural members.

Specific Retrofit Method:
Fuse cable connections.

Specific Retrofit Concept:
Use a specially fabricated turnbuckle eyebolt to fuse a cable connection.

Notes:
Fabricate eyebolt using material with highly predictable failure load.

Provide dimensional tolerances to fabricator.

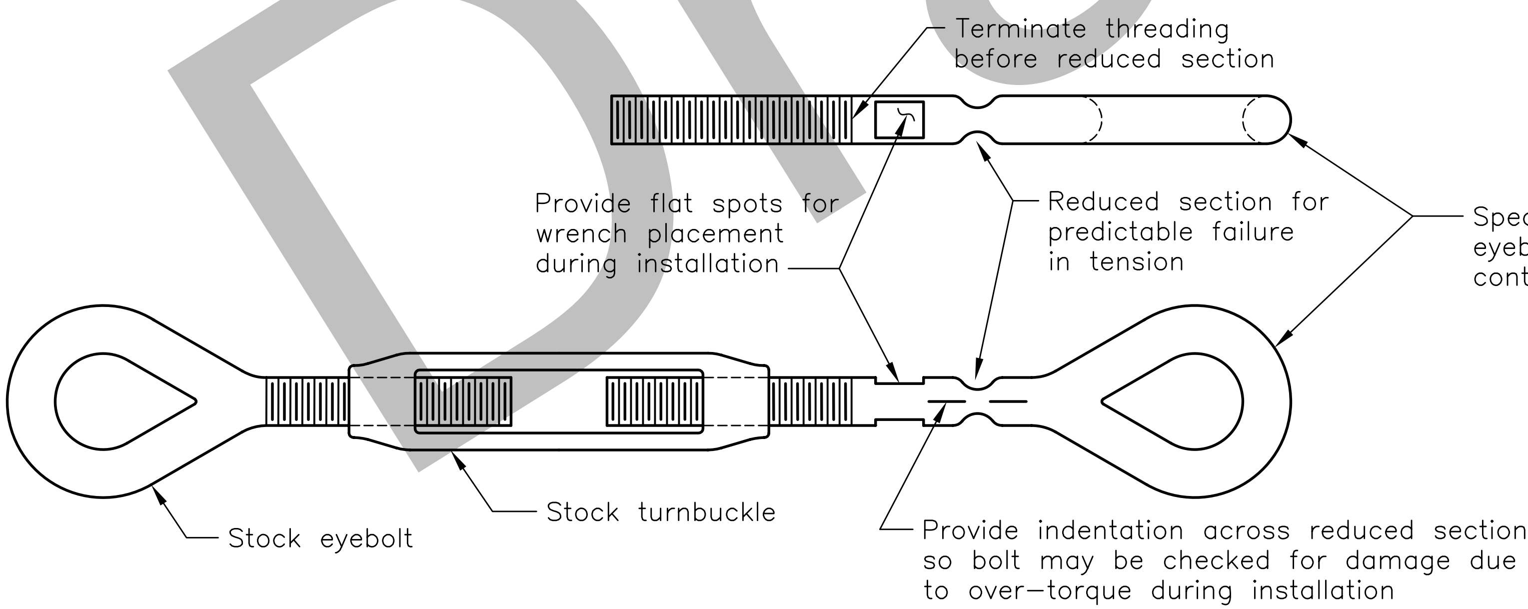
Ensure reduced section will produce the controlling load.

Pros:
Allows easy adjustment of cable lengths, while also functioning as a structural fuse.

Cons:
Requires custom fabrication.

Reduced section is susceptible to damage from bending and torque.

Analysis Issues:
Determination of acceptable dimensional and material—strength tolerances.



FUSED CABLE SPLICE

General Retrofit Method:
Provide a fused connection.

General Retrofit Principle:
Provide a fused connection to mitigate damage to structural members.

Specific Retrofit Method:
Fuse cable connection.

Specific Retrofit Concept:
Use a necked down plate as a fuse.

Notes:
Provide fabricator with dimensional tolerances.

Fabricate splice plate from material material with highly predictable failure load.

Ensure that the necked down section is the weakest part of the connection.

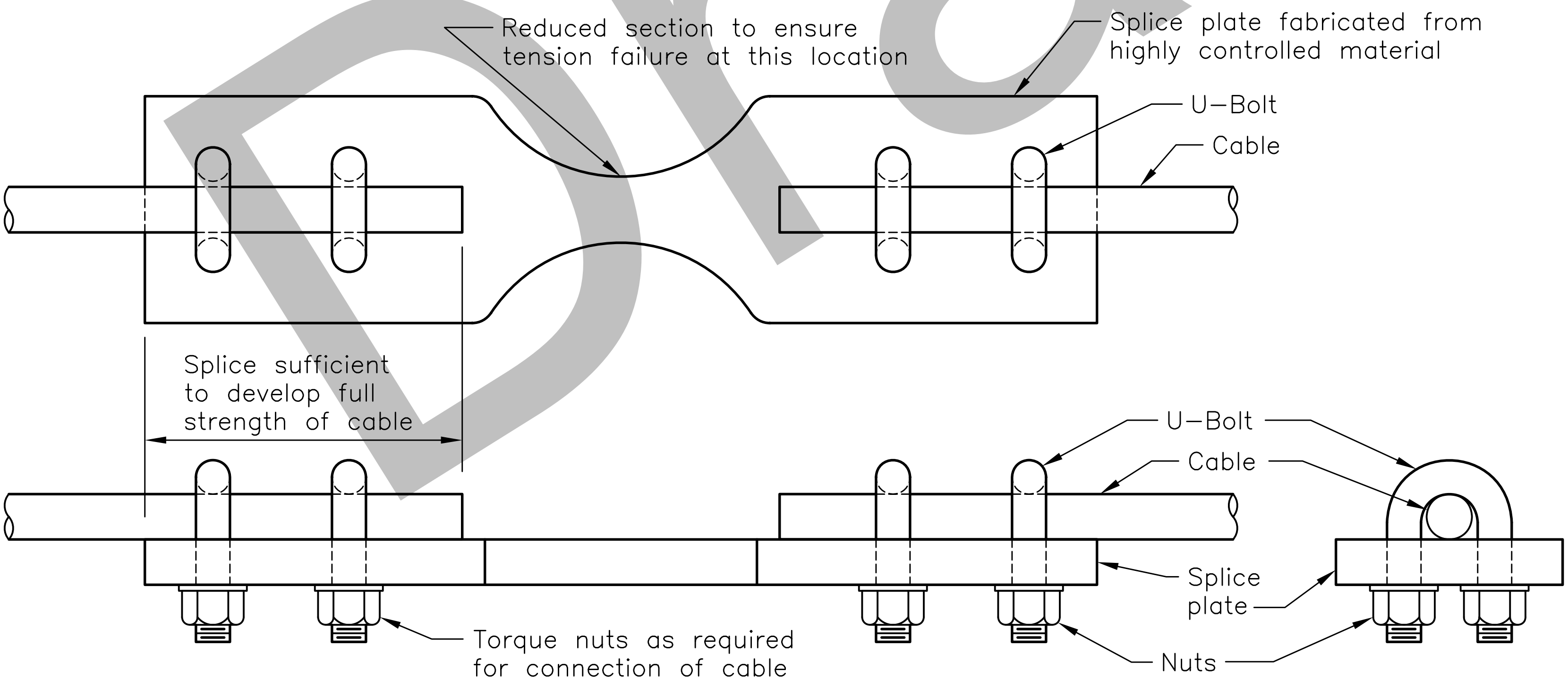
Pros:
Fabrication of splice plate is less complicated than that of turnbuckle components.

Cons:
Does not permit easy adjustment of cable length.

Requires custom fabrication.

Analysis Issues:
Determination of dimensional and material—strength tolerances.

Connection is eccentrically loaded.



FUSED SHEAR BLOCK

General Retrofit Method:
Provide a fused connection.

General Retrofit Principle:
Provide a fused connection to mitigate damage to structural members.

Specific Retrofit Method:
Fuse shear block to pier cap connection.

Specific Retrofit Concept:
Fabricate shear block so that shear is carried only by reinforcing, which is designed to act as a fuse.

Notes:
Prevent bond between existing pier cap and new shear block concrete to prevent transfer of shear through the interface.

Pros:

Cons:
Reinforcing may be susceptible to corrosion at the shear block to existing pier cap interface.

May only be used with new shear blocks.

Analysis Issues:
If multiple shear blocks are used the possibility that blocks will not be simultaneously engaged should be investigated when determining the required strength of the fuse.