Attachment G

Matrix of Parameters for Force Calculations Comparison

Comment	Case #	Environmental Conditions						Bridge Parameters					
		d_s (ft)	Significant Wave Height (ft)	Maximum Wave Height (ft)	Wave Period (sec)	Wave Length (ft)	Wave Crest Height* (ft)	Y_c (ft)	Span Width (ft)	Span Length (ft)	b (ft)	a (ft)	Number of Girders
I10-Escambia Bay Span and Conditions	123	37.1	4.7	8.2	4.1	92.3	4.95	1.6	35.3	60	0.5833	3.75	6
I10-Escambia Bay Span and Conditions	131	39.5	4.7	8.2	4.1	92.7	4.94	-0.3	35.3	60	0.5833	3.75	6
I10 - Lake Pontchartrain	W363	22.1	6.5	12.0	5.1	125.8	8.2	1.2	45.5	65	0.54	3.75	6
I10 - Lake Pontchartrain	W382	21.2	6.4	11.7	5.1	124.3	8.1	-3.8	45.5	65	0.54	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	1	40	4.4	8	3.1	58.7	5.27	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	2	40	4.4	8	5	128.3	4.48	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	3	40	4.4	8	8	254	4.55	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	4	40	4.4	8	3.1	58.7	5.27	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	5	40	4.4	8	5	128.3	4.48	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	6	40	4.4	8	8	254	4.55	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	7	40	4.4	8	3.1	58.7	5.27	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	8	40	4.4	8	5	128.3	4.48	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	9	40	4.4	8	8	254	4.55	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	10	25	4.4	8	3.1	58.2	5.38	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	11	25	4.4	8	5	118.6	4.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	12	25	4.4	8	8	217	5.16	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	13	25	4.4	8	3.1	58.2	5.38	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	14	25	4.4	8	5	118.6	4.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	15	25	4.4	8	8	217	5.16	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	16	25	4.4	8	3.1	58.2	5.38	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	17	25	4.4	8	5	118.6	4.77	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	18	25	4.4	8	8	217	5.16	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	19	40	6.7	12	4	95.1	7.66	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	20	40	6.7	12	6	175.1	7.06	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	21	40	6.7	12	8	259	7.25	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	22	40	6.7	12	4	95.1	7.66	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	23	40	6.7	12	6	175.1	7.06	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	24	40	6.7	12	8	258.6	7.25	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	25	40	6.7	12	4	95.1	7.66	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	26	40	6.7	12	6	175.1	7.06	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	27	40	6.7	12	8	258.6	7.25	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	28	25	5.6	10	4	89.1	6.37	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	29	25	5.6	10	6	154.8	6.35	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	30	25	5.6	10	8	221.2	6.77	3	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	31	25	5.6	10	4	89.11	6.37	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	32	25	5.6	10	6	154.8	6.35	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	33	25	5.6	10	8	221.2	6.77	0	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	34	25	5.6	10	4	89.1	6.37	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	35	25	5.6	10	6	154.8	6.35	-9	35.3	60	0.58	3.75	6
I10-Escambia Bay Span Hypothetical Conditions	36	25	5.6	10	8	221.2	6.77	-9	35.3	60	0.58	3.75	6

OEA Inc.											
Maximum Vertical Force				Maximum Moment about the Leading Edge				Maximum Moment about the Trailing Edge			
Maximum Vertical Force (kips)	Associated Horizontal Force (kips)	Associated Moment about the Leading Edge (ft-kips)	Associated Moment about the Trailing Edge (ft-kips)	Maximum Moment about the Leading Edge (ft-kips)	Associated Horizontal Force (kips)	Associated Vertical Force (kips)	Associated Moment about the Trailing Edge (ft-kips)	Maximum Moment about the Trailing Edge (ft-kips)	Associated Horizontal Force (kips)	Associated Vertical Force (kips)	Associated Moment about the Leading Edge (ft-kips)

Wallingford Method 1						Wallingford Method 2						Douglas Method				
Maximum Vertical Force						Maximum Vertical Force						Maximum Vertical Force				
Maximum Vertical Force (kips)	Associated Deck Horizontal Force (kips)	Associated Seaward Girder Horizontal Force (kips)	Associated Internal Girder Horizontal Force (kips)	Associated Moment about the Leading Edge (ft-kips)	Associated Moment about the Trailing Edge (ft-kips)	Maximum Vertical Force (kips)	Associated Deck Horizontal Force (kips)	Associated Seaward Girder Horizontal Force (kips)	Associated Internal Girder Horizontal Force (kips)	Associated Moment about the Leading Edge (ft-kips)	Associated Moment about the Trailing Edge (ft-kips)	Maximum Vertical Force (kips)	Associated Deck Horizontal Force (kips)	Associated Girder Horizontal Force (kips)	Associated Moment about the Leading Edge (ft-kips)	Associated Moment about the Trailing Edge (ft-kips)

Attachment H

Cataloged Observed Patterns of Failure

US 90 Henderson Point—No Vertical Restraint (if bond breaker used on dowels)

Information sources: Mississippi DOT pictures and bridge plans

General Information:

- Side by side chorded multi-span prestressed bulb-T girder bridges
- Bridge Drawings dated 1996
- Elastomeric bearing pads at free end
- Full depth doweled diaphragms at fixed ends
- One span of bridge dislodged completely, others shifted
- Dislodged span was where bridge met grade (lowest elevation/most submersion)
- Dislodged span was about 123' long and 48' wide

Information on full depth diaphragms with dowel bars:

- Dowel bars still intact (not sheared)
- Some dowels bent (from beams landing on them)
- Others still upright (at edge, where span moved away from them)
- Dowels in fixed diaphragm are double leg #5's at 12" (See bridge plan sheets 16 to 19, and 23)
- Some States use bond breakers between the steel dowel bars and the diaphragm concrete to enable jacking of the beams for bearing replacement and other maintenance
- No bond breaker details given in plans
- Do dowel bars provide any vertical restraint?

- Concrete in end diaphragm is damaged (due to rotation of span about fixed end or dropping of the span?)

Information on entrapped air:

- At least some cavities had drains through the deck that would have allowed air to escape from between the beams (roughly 8” by 3” openings spaced at 8’ 10”)
- Not all spans have the same drain layout
- The dislodged span had deck drains that would vent air from only one of the 5 air-spaces between girders (six beam cross section)
- The dislodged span was sloped longitudinally, so the amount of air trapped between beams and diaphragms is less than would be trapped had the section not been sloped, assuming water rises uniformly
- Change in elevation along the length of the dislodged span was about 2.85’, or about 1’ of elevation change between successive diaphragms
- Higher end has about 5’6” average airspace height, two other air pockets have about 5’0” average air space height. (full depth end diaphragm allows more air to be trapped than intermediate diaphragms, which were not full depth)

Failure Considerations:

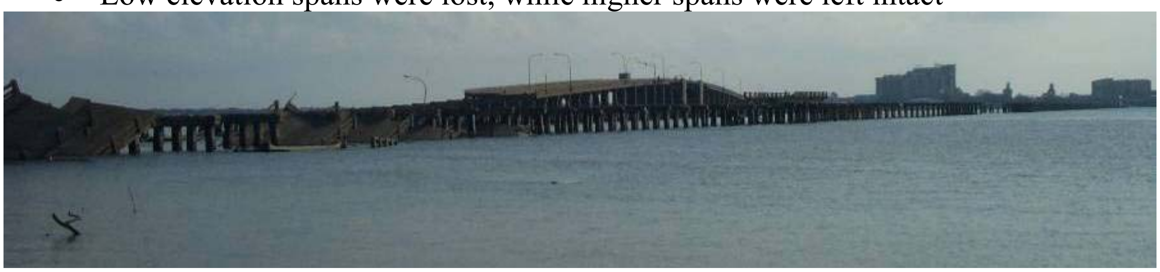
- Because there was no vertical restraint (assuming dowels ineffective) it is likely that the superstructure was lifted up and off of the dowels then was pushed laterally by waves, current, and/or wind.

US 90 Biloxi Bay / Biloxi-Ocean Springs—No Vertical Restraint

Information sources: Mississippi DOT Pictures and Bridge Plans

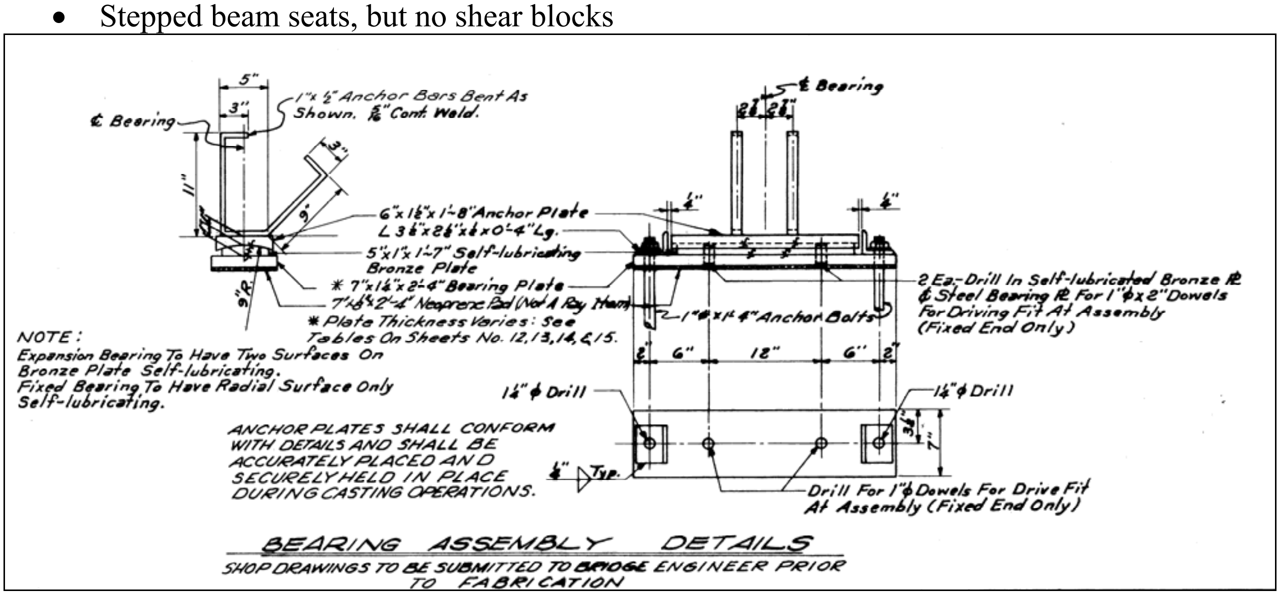
General information:

- Bridge contained concrete girder spans, steel girder spans, and a bascule span
- Bridge plans dated 1959
- Concrete girder spans
    - Prestressed girders
    - 52’ typical span length
    - 33’ 5” typical span width
    - Used on lower elevation spans
    - Majority of bridge was concrete spans
- Steel girder spans
    - Wide flange sections
    - 76’ typical span length
    - Used on higher elevation spans adjacent to bascule
- Low elevation spans were lost, while higher spans were left intact



Information on bearings for 52’ prestressed girder spans:

- Bronze bearings used (Bridge plans, sheet 19)
- No positive connection between beams and pier cap (uplift not resisted)
- Small angles used to restrain transverse movement of bronze bearings (2 ½” high)
- Stepped beam seats, but no shear blocks



Information on entrapped air for 52’ concrete spans:

- Deck had 4” inside diameter drains through the deck spaced at 11’ center to center.
- These drains would have allowed air to escape from only one of the 5 air-spaces between girders (six girder cross section)
- Solid end and intermediate diaphragms were present (diaphragms terminated 6” from bottom of beam)
- Neglecting weight of diaphragms and dead loads such as railings and light standards, assuming air escaped from one airspace due to vents, and assuming the water level to be at the top of the sidewalk, the weight of a typical 52’ prestressed span was
    - Un-submerged 323.6k
    - Submerged, air not compressed (ignore ideal gas law) 10.4k
    - Submerged, air compressed using ideal gas law, 1^{st} iteration on h 28.7k
    - Submerged, air compressed using ideal gas law, iterated h 27.7k
    - Where h is the height of the compressed air
- Sections very nearly buoyant when submerged in static water

Failure Considerations:

- Conditions that failed the bridge obviously did not involve static water
- Is it possible that the sections were displaced prior to complete submersion? (i.e. wave forces failed the bridge before the above-mentioned buoyancy calculations materialized)
- Likely superstructure was lifted (above the small angles or other features, such as stepped beam seats, that might have provided lateral or longitudinal restraint), then lateral and or longitudinal forces due to waves, tides, and/or wind pushed sections off of piers

US 90 Bay St. Louis—No Vertical Restraint (for concrete spans)

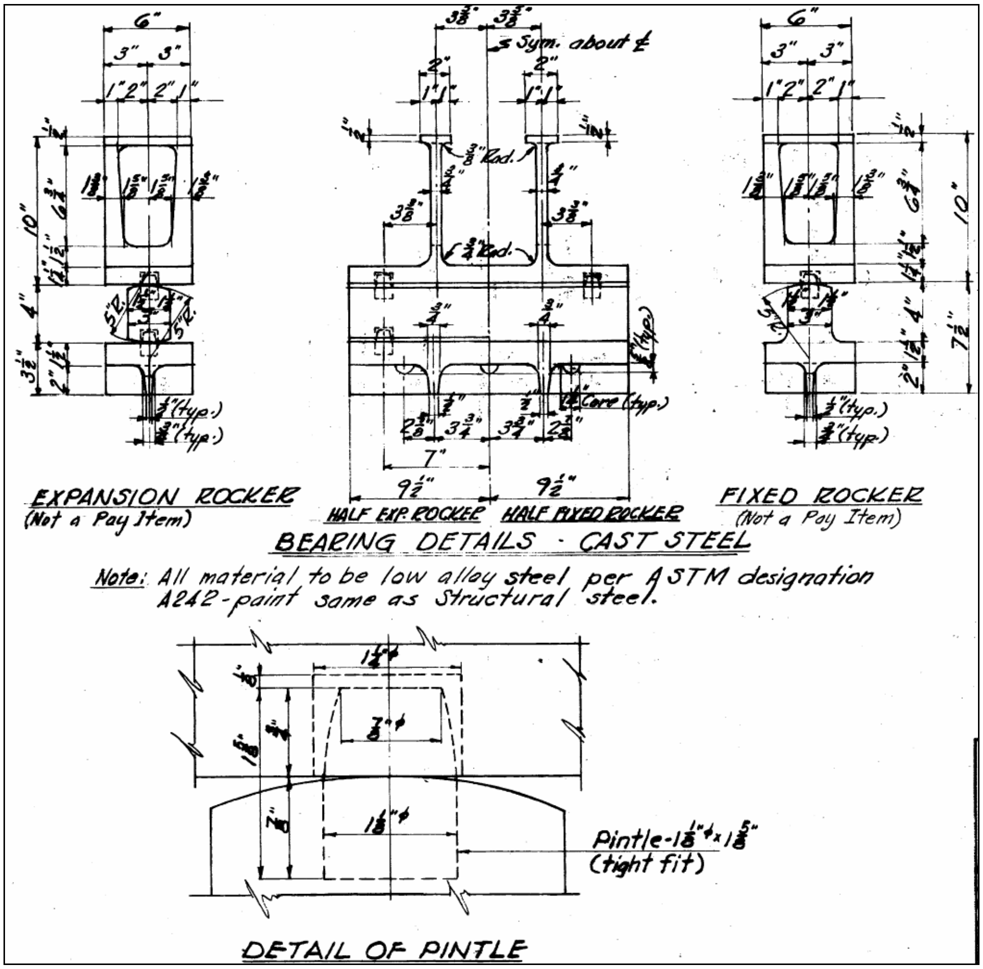
Information sources: Mississippi DOT Pictures, bridge plans, AASHTO slideshow, and pictures taken from paper by Robertson et al

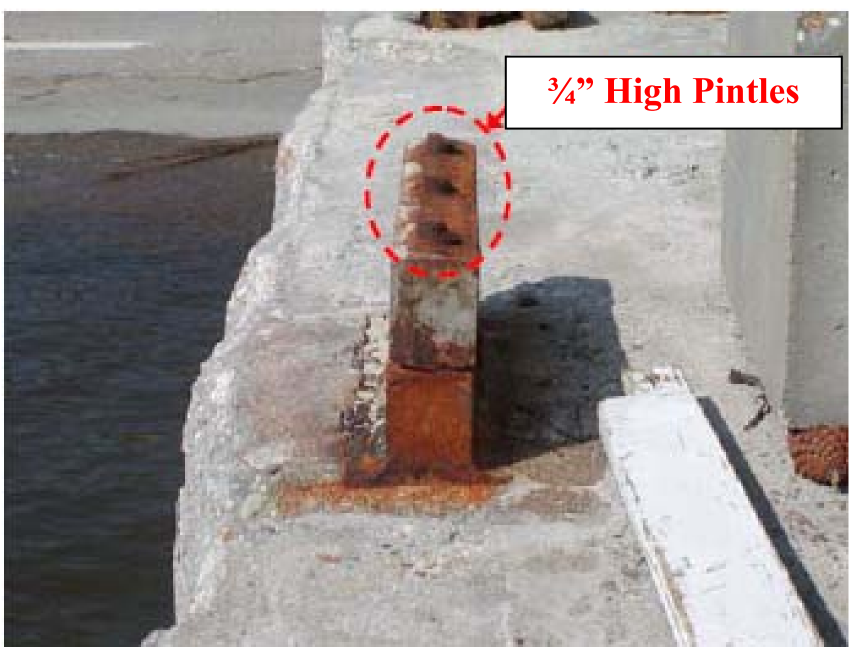
General Information:

- Bridge contained concrete girder spans, steel girder spans, and a bascule span
- Concrete girder spans
    - Reinforced concrete T-Beams
    - 41’ typical span length
    - Majority of bridge was concrete spans
    - Almost all dislodged
- Steel girder spans (two)
    - Wide flange sections
    - 75’ span length
    - Used as approach span on both sides of bascule
    - Concrete deck appears to have been non-composite (no shear studs)
    - One steel span dislodged
    - One steel span remained on piers, but the deck was missing
- Bascule span damaged, but not examined herein
    - Movement of bascule may provide an estimate of wave loads
- Storm surge elevation 24.9’, storm surge plus wave 40.2’ (AASHTO slideshow)
- Dislodged spans found as far as 220’ from their original location (AASHTO slideshow)
- Bridge plans dated 1951

Information on bearings for 41’ concrete spans

- Bearings provide no positive connection that would resist uplift
- Bearing details shown below are from sheet 12 of the bridge plans
- Pintles should project ¾” from the rocker or sole plate
- The picture below from Robertson et al shows Pintles that appear to be undamaged


EXPANSION ROCKER
(Not a Pay Item)
HALF EXP. ROCKER
HALF FIXED ROCKER
FIXED ROCKER
(Not a Pay Item)
BEARING DETAILS - CAST STEEL
Note: All material to be low alloy steel per ASTM designation A242 - paint same as Structural steel.
Sym. about ℄
Pintle - 1⅛"φ x 1⅝"
(tight fit)
DETAIL OF PINTLE


¾" High Pintles

Information on entrapped air for 41’ concrete spans

- 3” diameter drains through deck at 10’ increments
- Drains would permit air to escape from one of the three air-spaces between girders
- Diaphragms were only slightly more than half of the beam depth below the deck
- Comparatively small volume of trapped air

Information on steel spans:

- Bearings of steel spans provided vertical restraint in the form of bolts at both the expansion and fixed ends.
- It appears that the deck was not composite with the girders

Failure Considerations:

- Likely the concrete spans were lifted (above the Pintles or other features that might provide lateral or longitudinal restraint), then lateral and or longitudinal forces due to waves, tides, and/or wind pushed sections off of piers
- Steel spans had connections capable of resisting at least some uplift
- For the steel span that remained on the piers, it is likely that the non-composite slab was lifted away from the beams by wave forces while the beams remained in place because of the provided connections between the beams and pier
- It is unclear if the steel span that was dislodged was knocked off of the pier before or after the non-composite deck was separated from the girders.

Popps Ferry Bridge

Information Source: Mississippi DOT Pictures (no plans)

General Information:

- Prestressed bulb T girder spans
- Four beam cross section, bents have 4 columns with a cap beam
- It appears that the bridge was designed so that the beams would sit directly over the bent columns
- The spans were shifted, but not lost
- Details of bearings unavailable
- Shifting of the spans caused damage to the pier cap
- Prestressed beams show cracking in the bottom flange near an intermediate external diaphragm (it is unknown how many beams exhibited this behavior or if is was an isolated occurrence)
- Damage to end of beams in area of sole plate
- Failure of deck/sidewalk

Damage resulting from shifted spans

- Cracked pier cap ends
  - Beams came to rest at the end of pier caps
  - The pier cap of pier 52 looks like it sheared where reinforcement started (i.e. beam came to rest with 2” or 3” of bearing on end of pier cap and the cap failed in direct vertical shear along the plane where the first tie/stirrup was)
  - Shallow member beam theory is not applicable to the pier cap overhang due to the large depth to length ratio.

- Pier cap beam cracked between columns
    - Superstructure beams came to rest midway between columns
    - Pier cap beam shows inclined shear crack from face of column
    - Flexure cracks observed directly below some of the shifted superstructure beams.
    - Spalling under superstructure beam, insufficient bearing / bearing on corner???
    - Pier cap beam likely not designed to take superstructure reactions at these locations – beams supposed to sit directly over the columns


51


43

Cracked p/s beam in span / away from end -- Bottom flange at external diaphragm

- The overall context of the picture is not clear
- Other pictures indicate that there are intermediate diaphragms at midspan only
- Positive bending (tension cracks) when span picked up and dropped (or slammed)???
- Does not appear to be crushing from compression (negative bending from uplift or removal of the selfweight that counteracts the prestress force)

Deck/sidewalk failed

- Context of the picture is unclear
- Cantilevered section?
- Failed due to uplift on cantilever?

- Cracked beam end in area of sole plate
    - Due to shortened bearing length after span shifted?
    - Due to dropping of the span?

Failure Considerations:

- Damage to pier cap is due to movement of spans - prevent spans from moving to solve this problem
- Cracked beam near midspan diaphragm – more information needed, may be potential future specification issue
- Failed sidewalk – more information needed, may be a potential future specification issue (see also I-10 Twin Spans)
- Damage to beam near sole plate – prevent spans from moving to solve this problem

I-10 Twin Spans—Broken Connections

Information sources: "Hurricane Katrina, Performance of Transportation Systems" by ASCE, Bridge Plans, Survey of damage performed by Volkert and Associates

General Information:

- Bridge consisted of concrete spans and steels spans
- Concrete spans
  - Precast prestressed concrete girders
  - Typical span length 65 ft
  - Used steel and bronze bearings
- Steel Spans
  - Used at higher elevation, not impacted by surge
- Low lying spans impacted by surge while higher elevation spans largely undamaged
- Some spans completely dislodged while others shifted laterally

Information on bearings for concrete spans:

- Steel and bronze bearings used (details from bridge plans not included due to poor image quality)
- Only fixed end exterior girders had vertical restraint (two connections per span)
- Uplift load path from beam to pier is as follows
  - Two straps transfer force from the beam into the bronze plate (similar to US 90 Biloxi Bay Bearings, see provided details for that bridge)
    - The straps each have two legs, which are hooked and embedded into the beam
    - The straps are attached to the bronze plate through an unknown connection (shop drawings not available)
  - Two bolts transfer the force from the bronze plate into to the pier
    - The anchor bolts are embedded in the pier cap
- Area of steel for bolts and straps are about the same
  - Two 1" diameter bolts = 1.57 square inches

- Four legs of 1” x 3/8” bar = 1.50 square inches
- Steel strengths not given
- Bearings not having vertical restraint had lateral restraint in the form of small angles

Fixed bearing at an exterior beam - Bronze plate still attached to pier
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - (Bronze plate is still attached to pier, not shown)
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - (Bronze plate is still attached to pier, not shown)
Straps holding the bronze plate to the beam failed while the anchor bolts remain intact

Fixed bearing at an exterior beam - Bronze plate is still attached to the beam
Straps holding the bronze plate to the beam are intact while the anchor bolts failed

Free end any beam or fixed end interior beam (locations where plate not bolted to pier)
Bronze plates remain attached to the beam through the straps (expected behavior)

Cracks and spalls at beam ends

- Beams banging on pier?
- Shifted beams being supported with insufficient bearing area/surface?
- At locations where the bronze plates were anchored to both the beam and the pier, the beam to plate connection may have caused damage to the concrete?

Cracking/spalling/missing concrete in top of girders/deck

- Context of pictures unclear
- Negative bending--tension?
- Positive bending—compression—picking up and dropping (or slamming) of span?

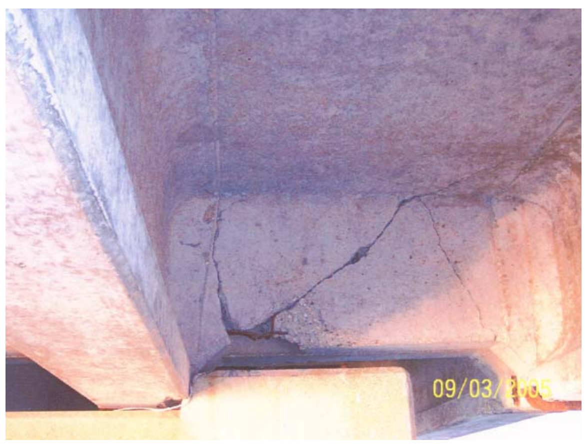
Damage to end of decks, ends of parapets, and ends of curbs

- Longitudinal movement caused banging?
- Spans become misaligned as they move sideways and get pinched between spans on either end (free end moves first)?

45 degree cracks at ends of decks

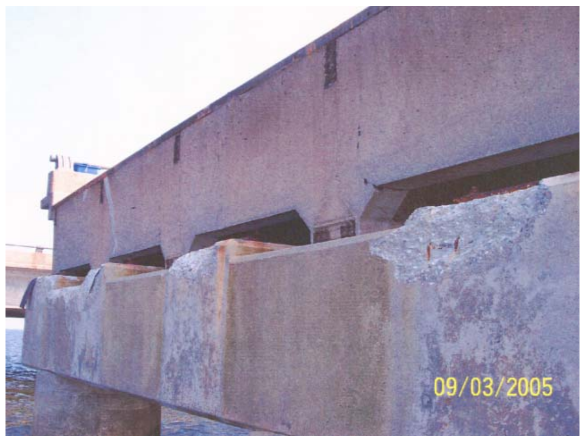
- Initiated through end diaphragms

- Beams gets caught on a stepped bearing seat as it moves sideways, halts sideways movement, tension transferred through diaphragm, cracks initiate and propagate into deck??
- Span shifts so that fascia beam is no longer supported by pier, weight of unsupported beam is now transferred through diaphragm which develops an inclined shear crack that propagates through the deck??



Spalls on pier, cracking at ends

- Insufficient bearing area when girder shifted??
- Dropping or banging of spans??
- Bending or direct shear on cantilevered bent cap



Missing barrier rail

- Cause of top of girder damage listed above?

Failure considerations:

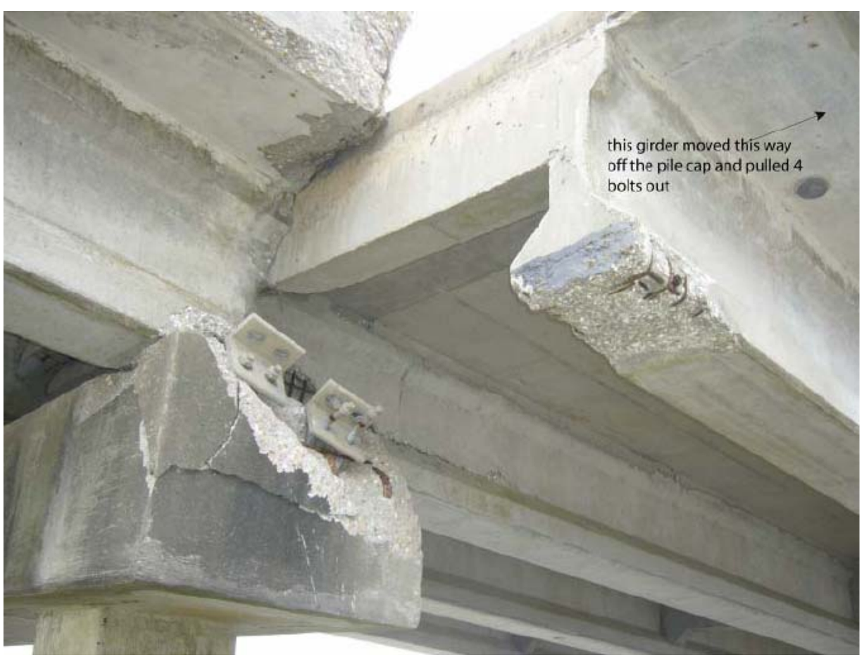
- Connections inadequate to prevent uplift of spans
  - Vertical restraint provided only at one end of each span
  - In some cases bolts connecting bronze plate to pier broke while in other cases the straps connecting the bronze plate to the beam broke
  - Area of bolts and straps about the same (steel strengths unknown)
  - About 3 square inches of steel to anchor each span
  - Provided area of steel reduced by corrosion?
  - It is unclear if the connections failed in shear, tension, or a combination of shear or tension
- Once spans were lifted (above any existing features that might provide lateral or longitudinal restraint), they moved laterally or longitudinally causing most of the damage pictured above, if spans could be anchored most damage would be avoided.
- 45 degree cracks in ends of decks which were initiated through the diaphragms are a potential future specification issue
  - If superstructure is restrained vertically by some adequate connection and shear blocks are used to restrain the superstructure laterally, will the diaphragms crack and lead to damage of the deck?
  - Are diaphragms an adequate way to anchor the superstructure?
  - Would different reinforcing details solve this problem?
- Damage to barrier rail is a potential future specification issue
  - Use of open rails
  - Use of more robust reinforcement
  - Consideration of uplift on cantilevered section of deck for quantity and development of reinforcement
  - How does the reinforcement used in this bridge compare to current design standards?
  - Are modern designs still susceptible to this type of failure?
  - See also Popps Ferry Bridge (a 1996 design)
- Cracking/spalling/missing concrete in top of girders/deck is a potential future specification issue
  - What caused this?

I-10 On-ramp at Midbay Crossing of US-90/98 – Broken Connections

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE and pictures in "Wave forces on Bridge Decks" by Douglass

General information

- Prestressed concrete simple spans 45' in length
- Built in 1970's
- Beams anchored using 1" diameter anchor bolts and 6" x 9" x 1" steel angles (ASCE)
- Concrete around bolts broke (Douglass et al.)
- Broken anchor bolts (ASCE)
- Some spans on curve were pushed toward center of the curve, so this may have prevented them from being fully dislodged

Pontchartrain Causeway—Limited Information

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- Construction generally similar to the I-10 Twin Spans
- Most of spans above surge level, thus undamaged
- The "turnaround" spans were at a lower elevation and 17 were lost, no specifics given

US-11 over Lake Pontchartrain—Largely Undamaged—Limited Information

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- Largely undamaged
- Haunched continuous girders
- Air vents in diaphragms
    - Where did these go, to an expansion joint??
    - How far did air have to travel to escape??
- What type of bearings were used?

LA-1 over Camanda Bay – Limited Information

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- Simply supported reinforced concrete
- 13 shifted spans
- Spalling and exposed rebar from debris
    - Would a sacrificial barrier to protect the structure be practical?
- No mention of bearing types

David V. LaRosa Bridge – Limited information

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- No specifics given
- A few shifted spans

Precast Bridge at Bayou La Batre—Banging of Adjacent Boxes

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- Single span precast girder bridge

- Prestressed adjacent box beams
- No cast in place deck or diaphragms
- Spalling between beams caused by them banging into one another
- No cast in place shear keys, or cast in place deck, or post tensioning of the beams transversely to prevent rattling or banging of beams??

Dauphin Island Bridge –New Bridge Undamaged – Limited information

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- First bridge destroyed by hurricane Fredrick in 1979
- New bridge built using precast segmental construction
- Minimal damage to bridge (was this bridge above the surge?)
- Damage to approaches and fenders

Cochrane-Africatown USA Bridge – Vessel Impact

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- Cable stayed bridge
- Damaged cable when bridge was struck by oil platform

Biloxi Back Bay Bridge –Vessel Impact

Information source: "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- High-span bridge (remained above surge?)
- Impacted by barge, which damaged a pile/bent
- Superstructure intact

Ocean Springs Pascagula—Vessel Impact

Information source: Mississippi DOT pictures and "Hurricane Katrina, Performance of Transportation Systems" by ASCE

- 13 to 18 feet of storm surge
- Repeatedly impacted by barges
    - One barge carried large cranes
    - One or more smaller barges
    - Also a tugboat
- Damaged piers (lateral forces failed rigid frame)

- A continuous span section (six spans) shifted 45 inches
- Damaged fascia girders
    - Spalling
    - Exposed strands
    - One completely destroyed

Seems to be undamaged by storm surge, possibly because of continuity of girders

Attachment I

Retrofit Options

TENSION PILES

General Retrofit Method:
Anchor spans to new auxiliary foundation.

General Retrofit Principal:
Where the existing substructure has inadequate capacity to resist wave forces, anchor the spans to a new auxiliary foundation.

Specific Retrofit Method:
Anchor spans using 4 tension piles, 2 on each side of the bridge.

Specific Retrofit Concept:
Extreme uplift will begin to lift the superstructure from the existing piers. When this happens the tension piles will be engaged and downward reactions will be applied at the location of the tension piles.

Notes:
Place Tension Piles as to:
- Not interfere or interact with existing substructure.
- Not interfere with navigation.
- Produce stresses in the superstructure due to extreme uplift that are as favorable as possible.

Pros:
Suitable for low elevation spans where structure meets grade (structure cannot be raised above surge level).

Utilization of tension piles to resist lateral loads may also be possible. This will require battering the piles.

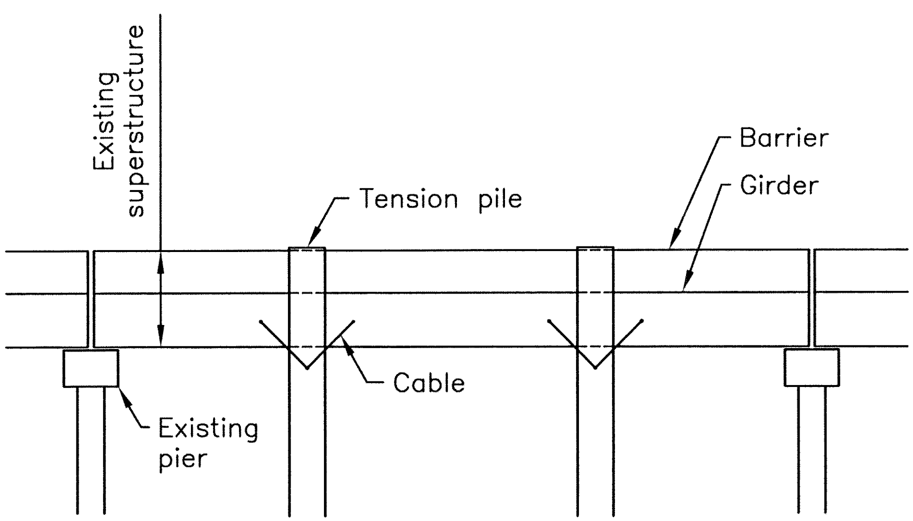
Cons:
Possibility exists for navigational, environmental, or scour issues.

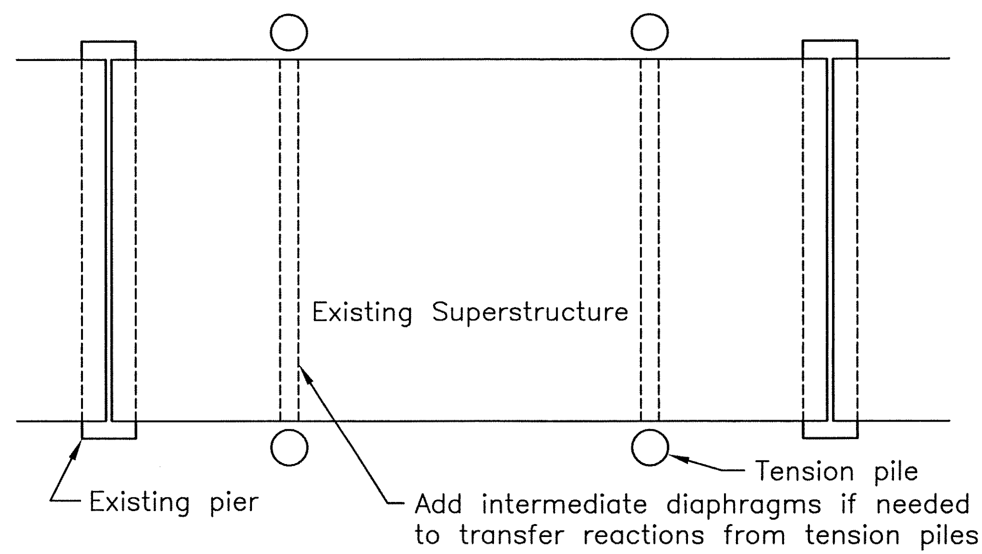
Analysis Issues:
Assuming uniformly distributed uplift (W) acting on the entire span, placing the tension piles at the quarter points of the span will result in zero bending moment at the midspan and a positive bending moment of $WL/32^2$ at the location of the piles.

Investigation of superstructure moments and stresses should consider the possibility of uplift on only part of the span.

Other pile locations should also be investigated as needed.

Tension piles will need adequate resistance to uplift, proprietary systems such as "spin fin piles" (see attached photos) may offer suitable solutions.





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 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 occurance of vehicle wheels passing over the vents.

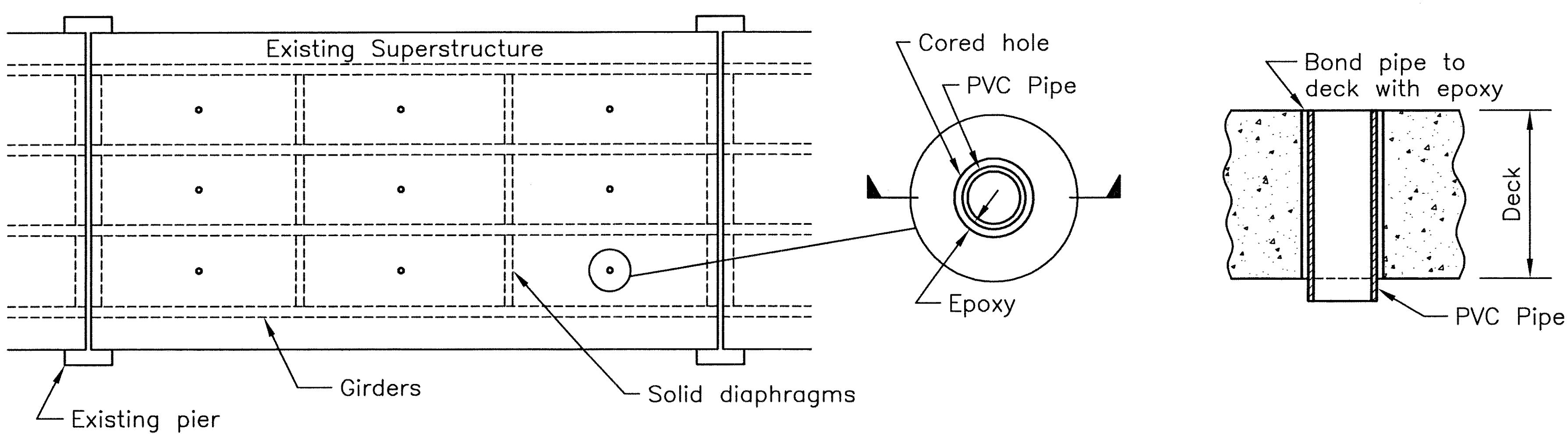
Diameter of vents to be based upon volume of trapped air and wave characteristics.

If possible place vents to clear reinforcement.

Pros:
Application simplicity.

Cons:
May not vent air fast enough.

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



STEEL WEB PLATES

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 plates 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 plates while <u>not</u> transferring loads under typical operating conditions.

Plates Transmit:
- Vertical shear as one beam lifts.
- Horizontal shear as one span shifts laterally.
- To transmit longitudinal forces through plate, size slots so that bolts engage plate after one span has displaced longitudinally an acceptable amount.
- To avoid transmission of longitudinal forces through plate, use long slots.

Notes:
Slots Permit free expansion and rotation under normal conditions.

Stiffen steel plate as needed to prevent buckling.

Locate bolts as far from end of beam as practicable to reduce drilling in high stress areas.

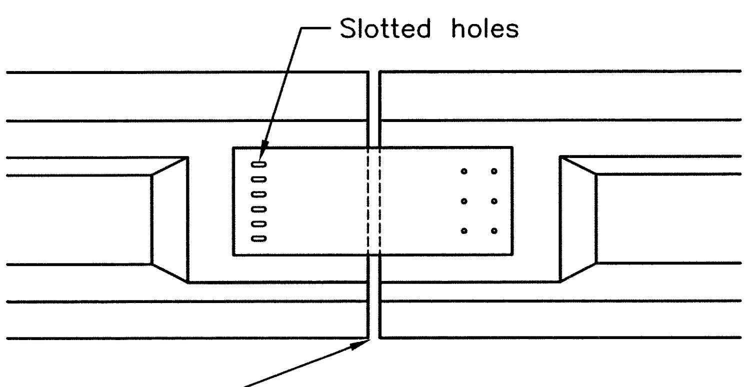
Locate bolts to clear reinforcing and prestressing.

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

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

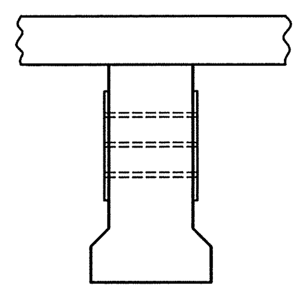
Beams without enlarged end cross section will be more sensitive to drilling

Analysis Issues:

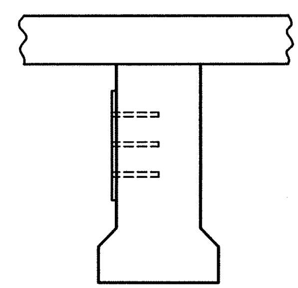


NOTE:
Place neoprene material between ends of beams to prevent spalling if contact is made, if desired

STEEL WEB PLATE ELEVATION



PLATES ON BOTH SIDES OF BEAM
(No end diaphragms)



PLATES ON ONE SIDE OF BEAM
(Facia beams with end diaphragms)

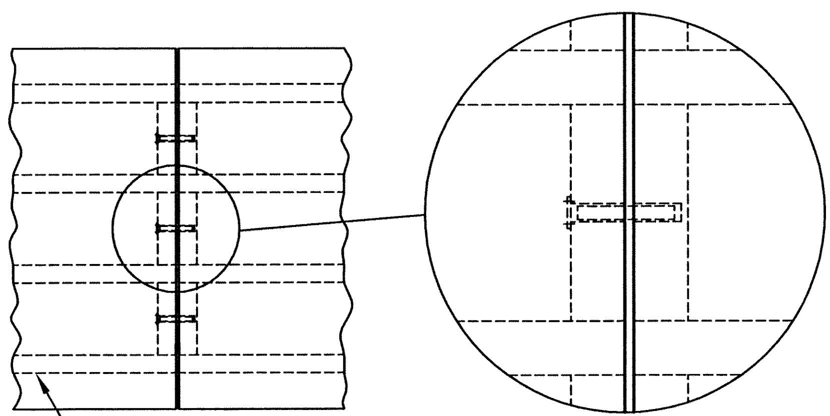
STEEL DIAPHRAGM PIPES

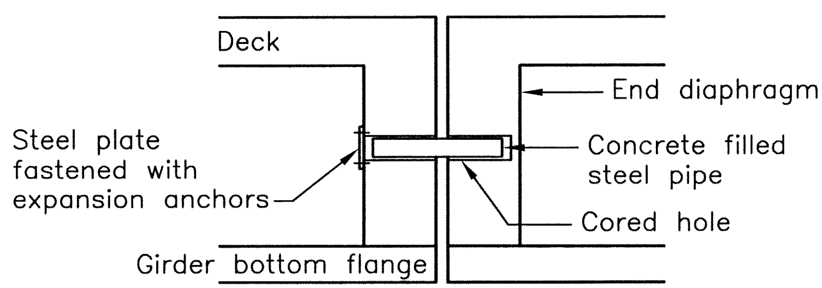
General Retrofit Method:
Tie spans together.

General Retrofit Principle:
When one span is experiencing maximum loading, adjacent spans will/may experience loads with a smaller magnitude. Tieing 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.

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





Pipes Transmit:
- Vertical shear as one span lifts.
- Horizontal shear as one span shifts laterally.

Notes:
Ensure length of pipe is less than length of cored hole so that longitudinal forces are not transferred through the connection.

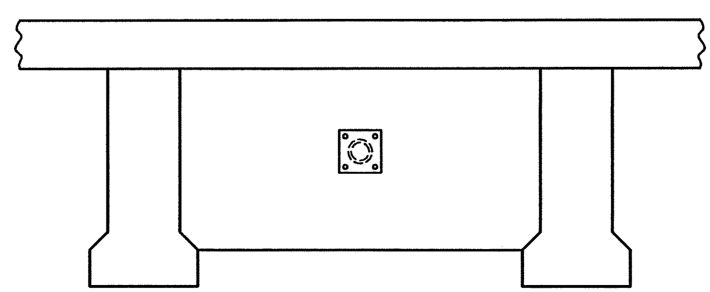
Multiple connections per diaphragm may be used if needed.

Pros:
Does not require drilling of beams.

Transmits lateral and vertical loads.

Cons:
Does not transmit longitudinal forces.

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



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.

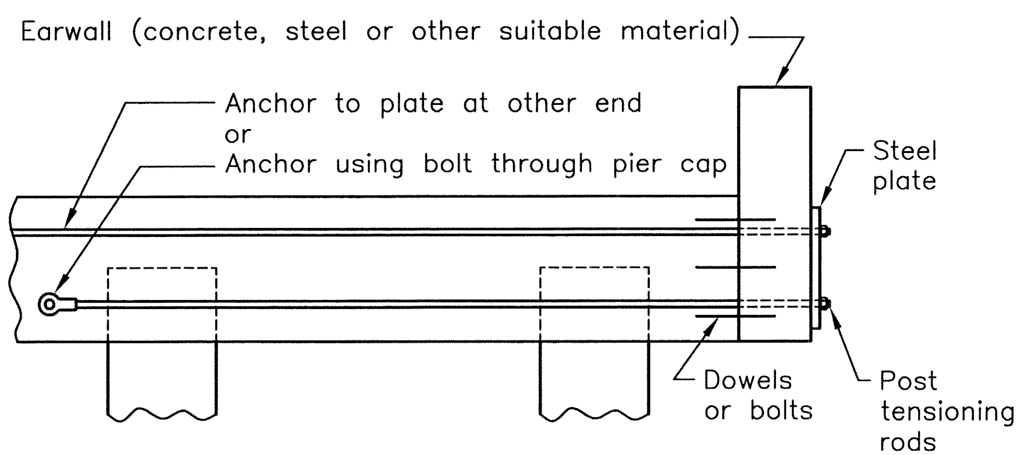
Specific Retrofit Concept:
Transfer lateral wave loads from fascia beams into pier cap through earwall.

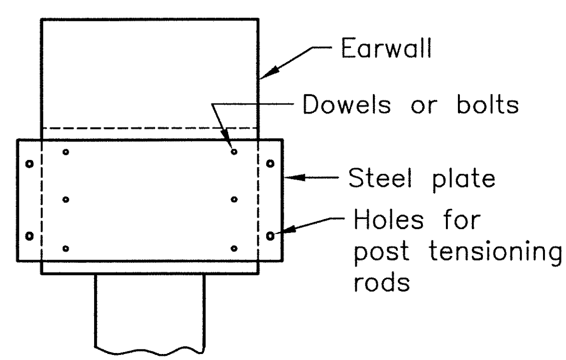
Notes:

Pros:

Cons:
Provides only lateral restraint.

Analysis Issues:





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.

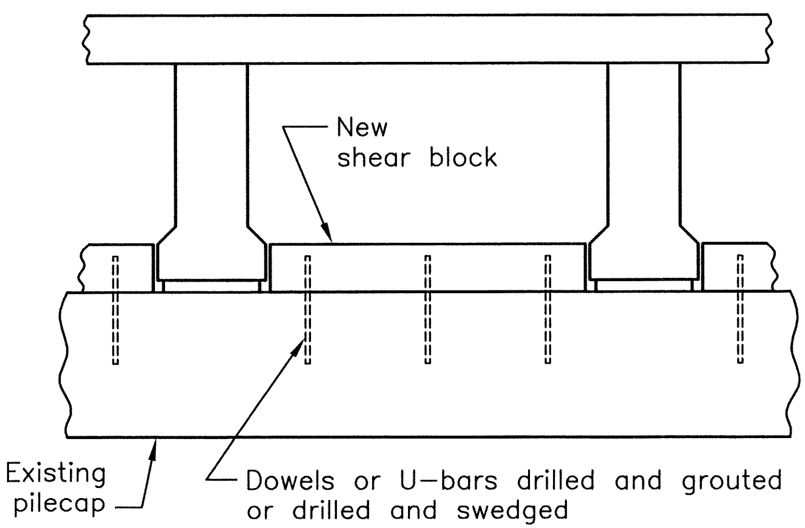
Pros:
High shear capacity may be obtained.

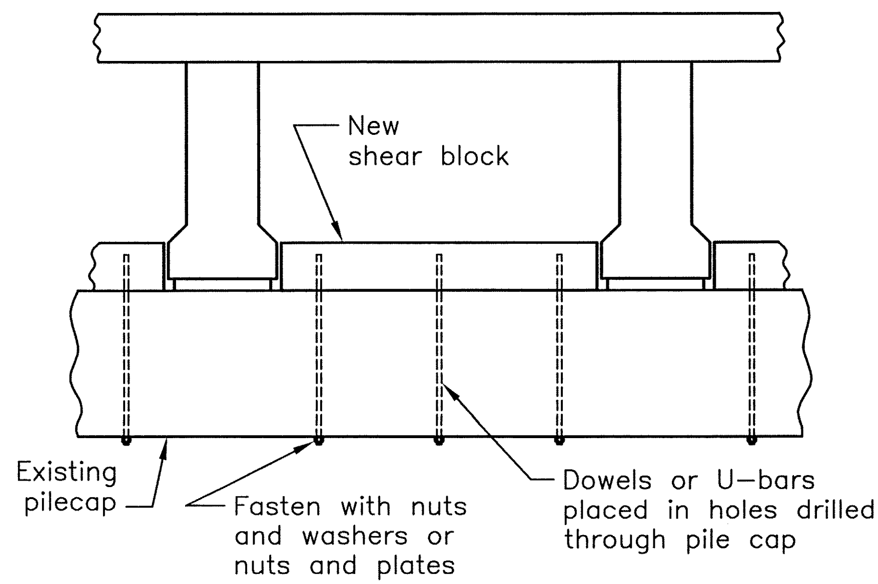
Does not require drilling of beams.

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

Provides only lateral restraint.

Analysis Issues:





HIGH SHEAR BLOCKS

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.

Pros:
High shear capacity may be obtained.

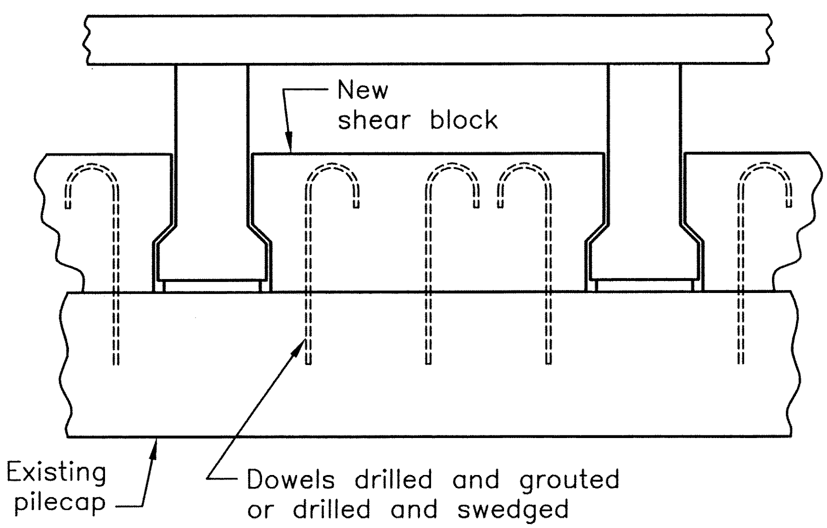
Does not require drilling of beams.

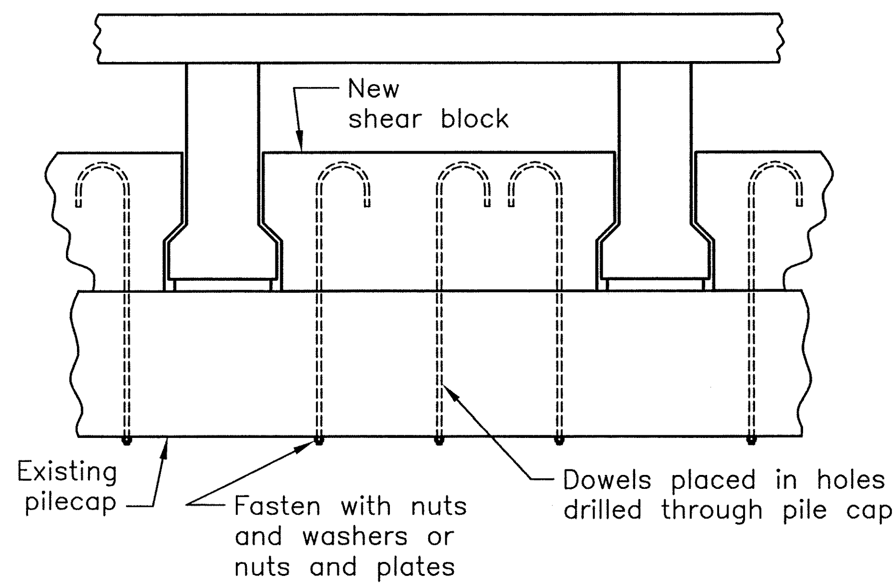
Provides lateral and vertical restraint.

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

Does not allow jacking of superstructure.

Analysis Issues:





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:

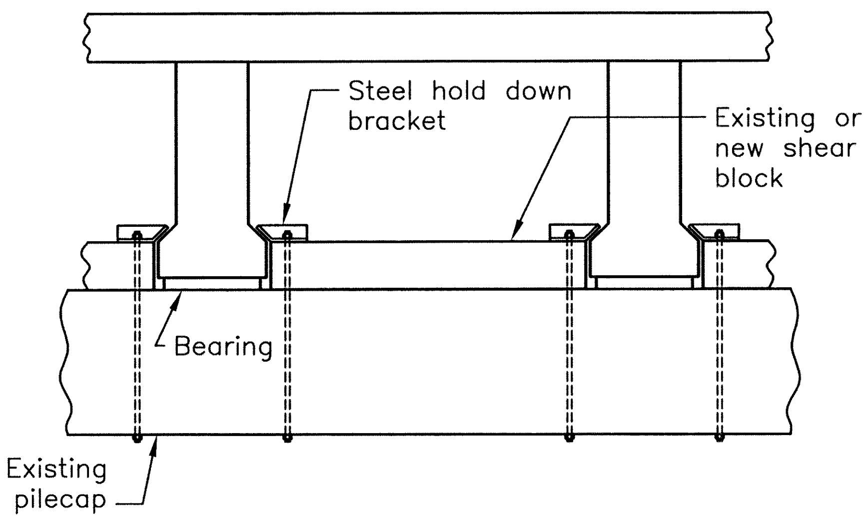
Pros:
May be used with new or existing concrete shear blocks.

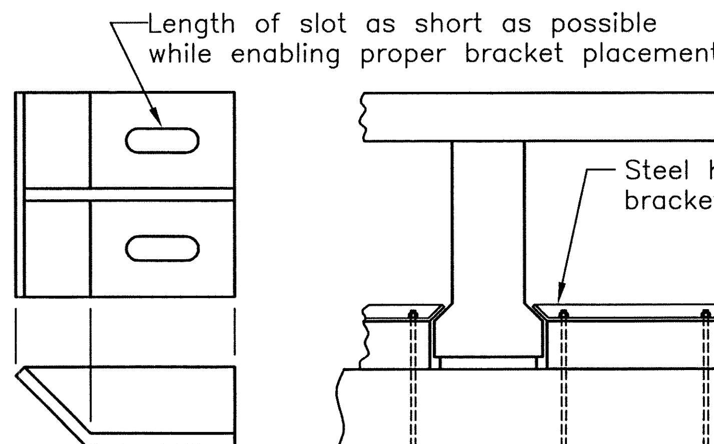
Does not require drilling of beams.

Both lateral and uplift loads are resisted.

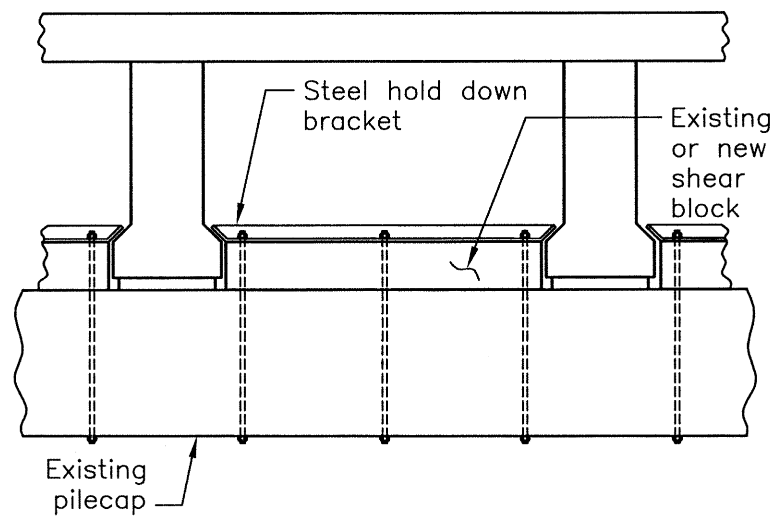
Cons:

Analysis Issues:





STEEL HOLD
DOWN BTACKET



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.

Notes:

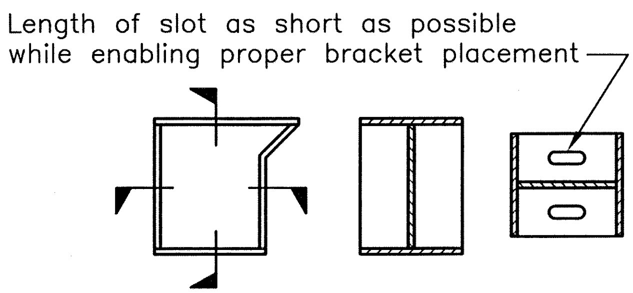
Pros:
Both lateral and uplift loads are resisted.

Does not require extensive field fabrication or concrete placement.

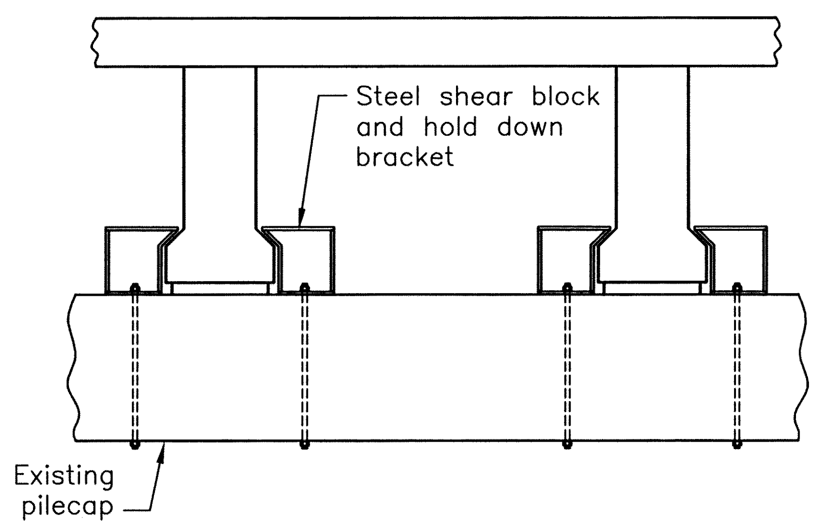
Does not require drilling of beams.

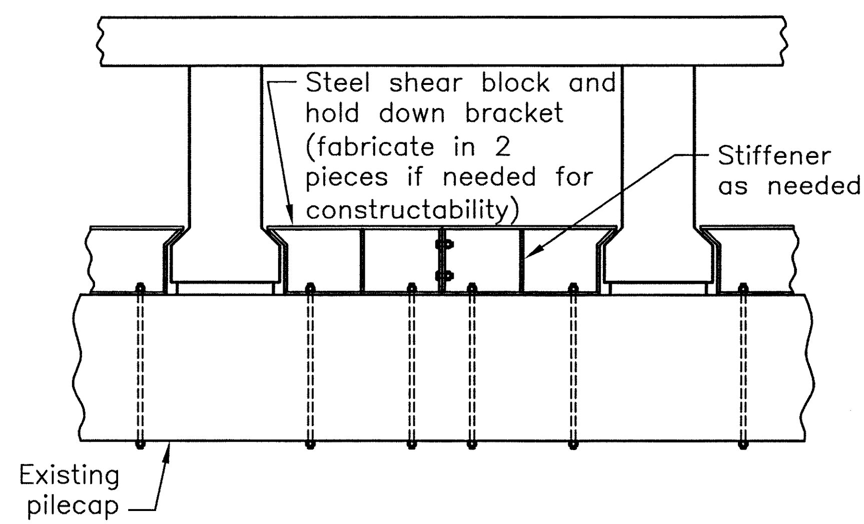
Cons:

Analysis Issues:



SHEAR BLOCK AND HOLD DOWN BRACKET





BEAM 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.

Pros:

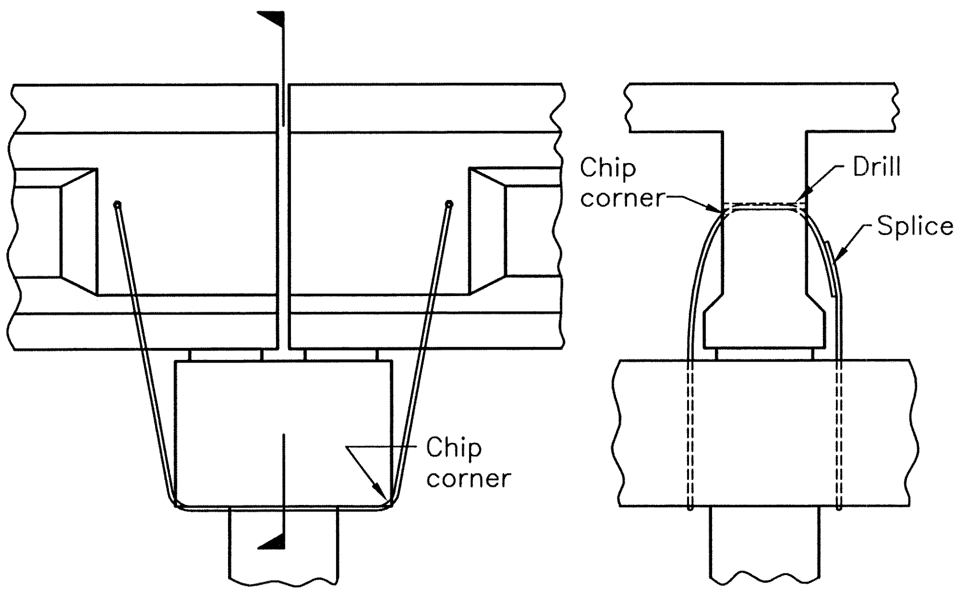
Requires minimal modification to pier cap beam.

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

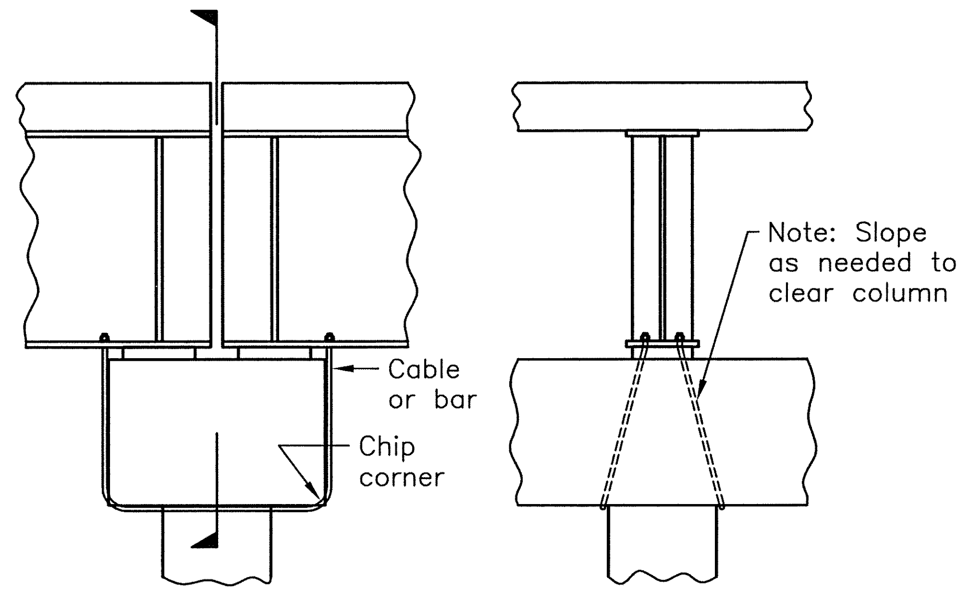
Cons:

Requires drilling of beams.

Analysis Issues:



CONCRETE



STEEL

DIAPHRAGM 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.

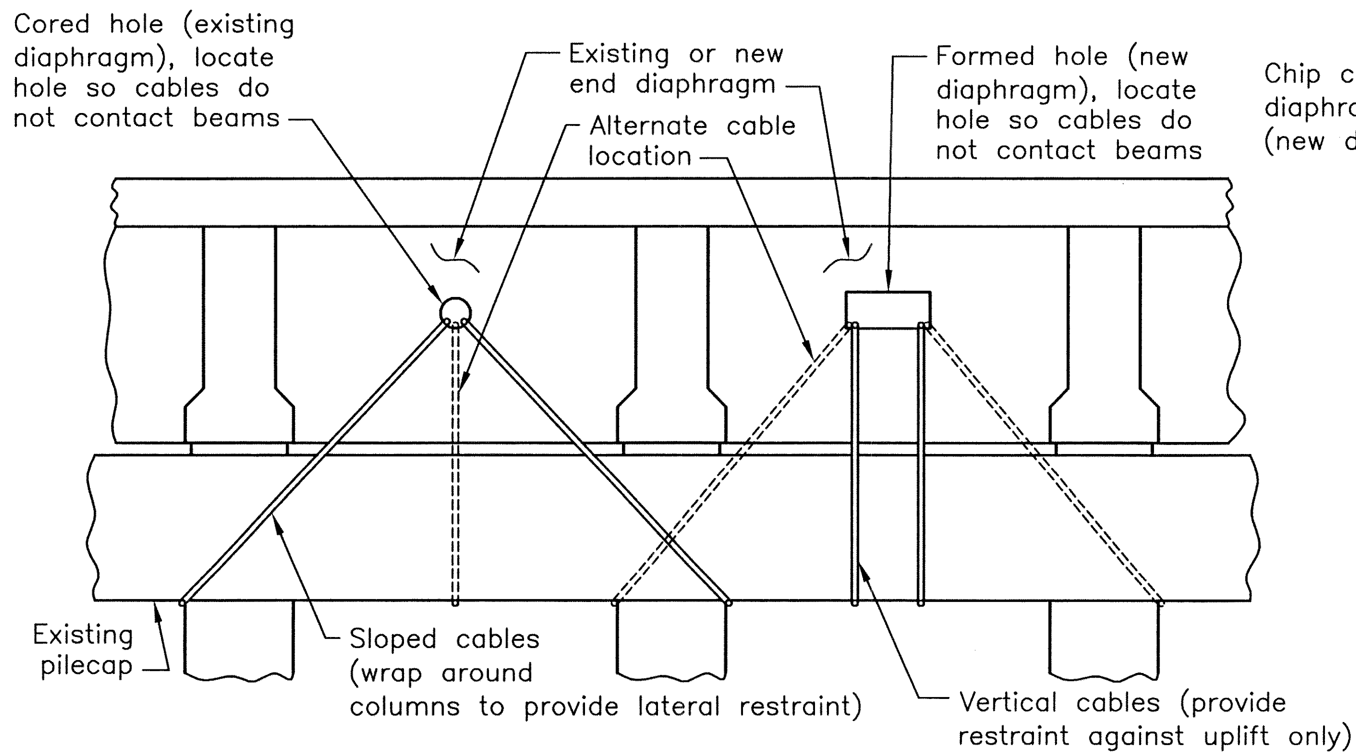
Does not require drilling of beams.

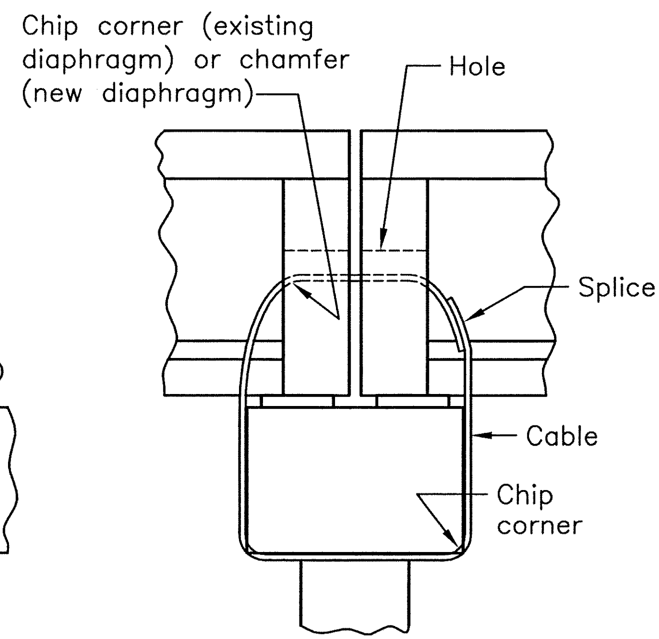
May be used when face of end diaphragm is or is not flush with pier cap.

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.





STEEL ANGLES/CHANNELS

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 angles or channels to connect end diaphragms to pier cap.

Specific Retrofit Concept:
Transfer lateral and uplift wave loads from the end diaphragms to the pier through steel angles.

Notes:

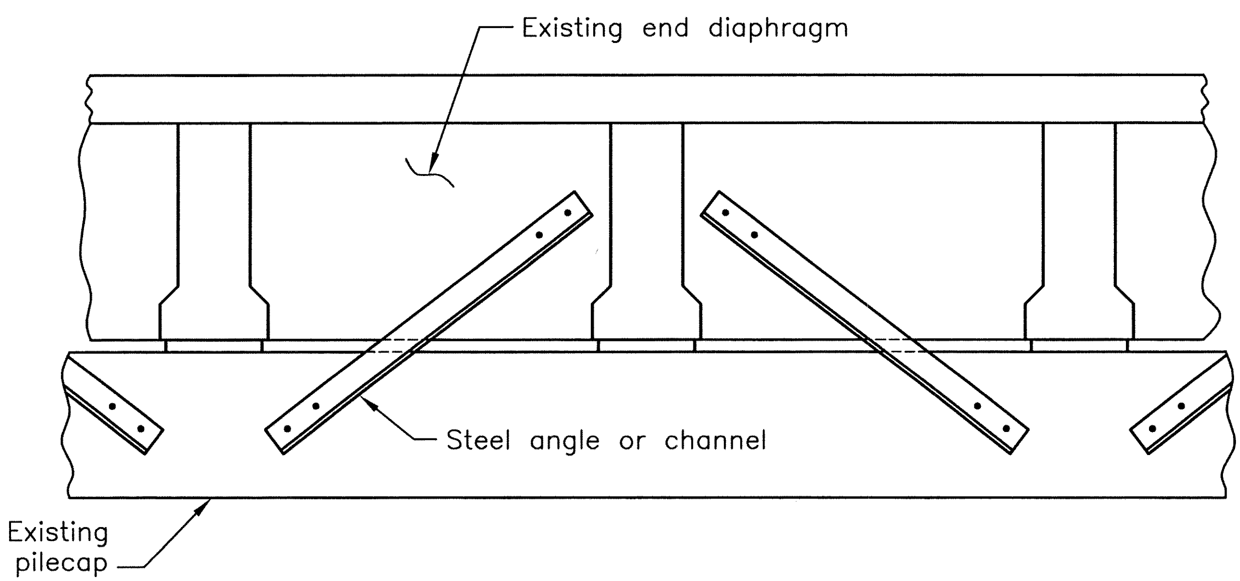
Pros:
Resists both uplift and lateral load.

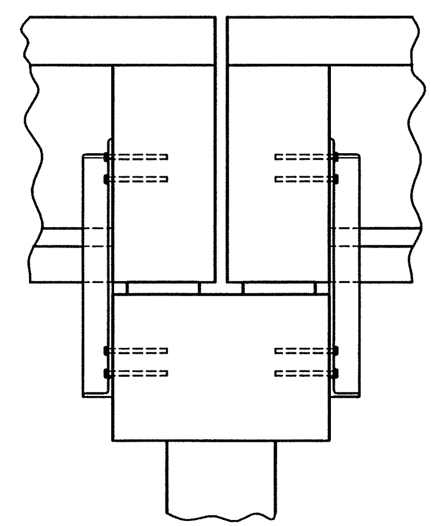
Does not require drilling of beams.

Cons:
Face of end diaphragm and pier cap beam must be flush for angle to be properly aligned or shimming must be added.

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

The effect of longitudinal expansion and contraction of the girders on the connection should be investigated.





STEEL BARS

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 bars to connect end diaphragm to pier.

Specific Retrofit Concept:
Transfer lateral and uplift wave loads from the end diaphragms to the pier through steel rods.

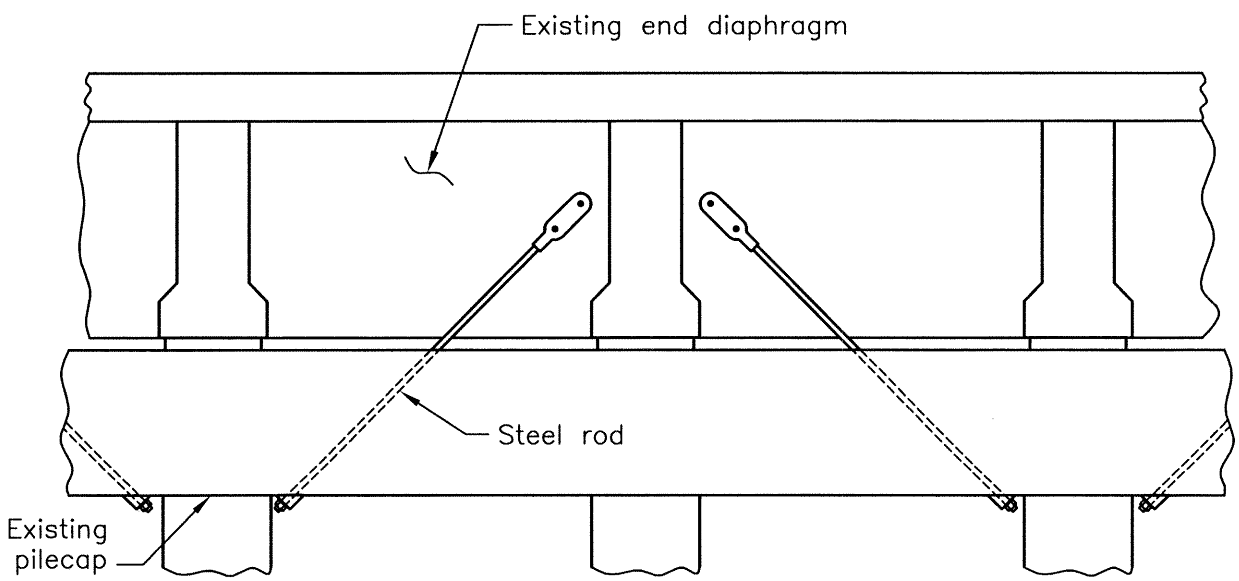
Notes:
Use oversized holes in pier cap (or other methods) to ensure that the connection is not damaged by longitudinal expansion of the girders.

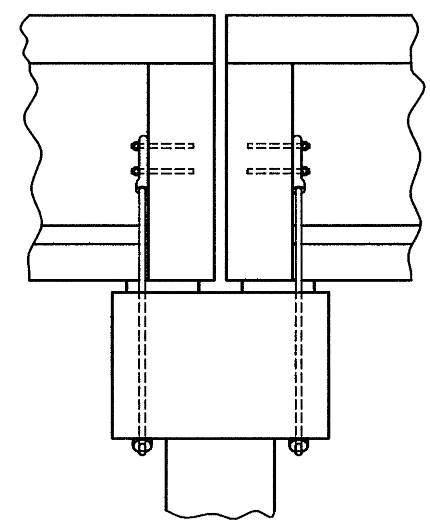
Pros:
Resists both uplift and lateral load.

Does not require drilling of beams.

Cons:
May only be used when face of end diaphragm is not flush with pier cap and the steel rods will be inside of the pier cap reinforcement.

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





PILE/COLUMN/PIER CAP STRENGTHENING

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.

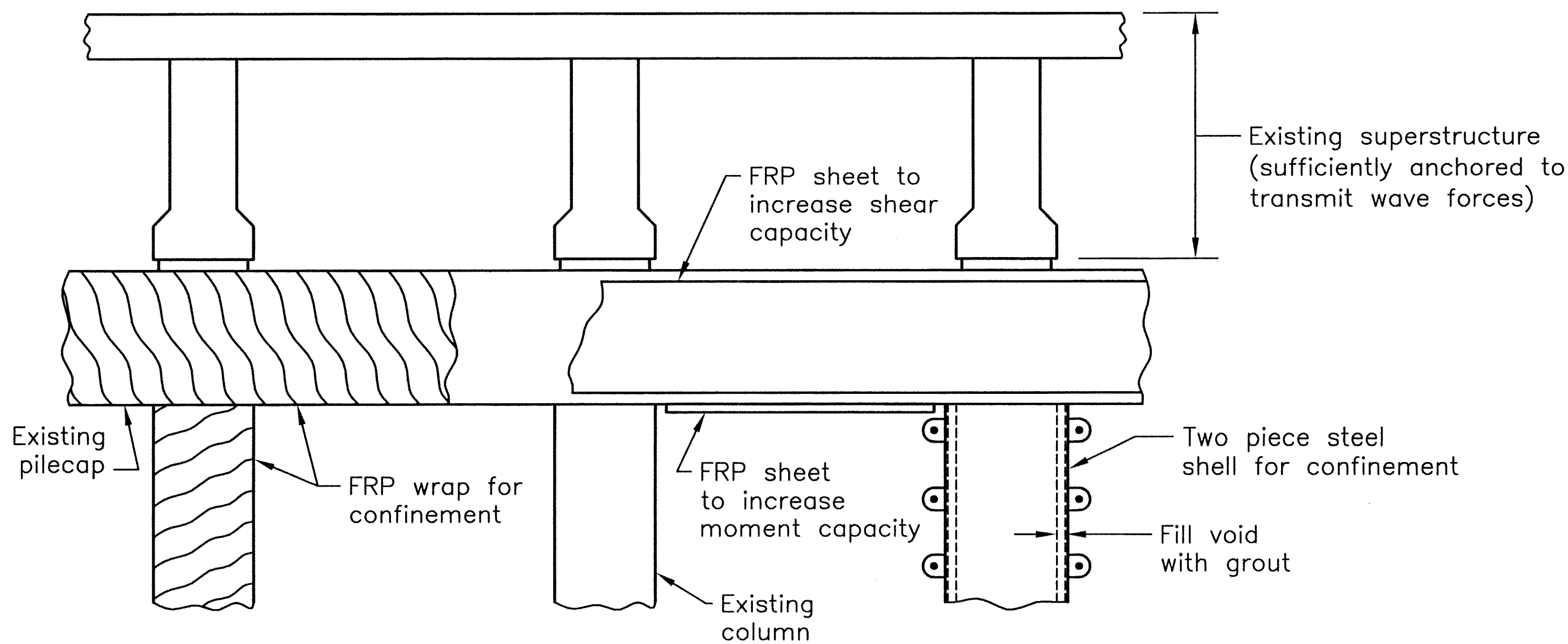
Specific Retrofit Concept:
Strengthen pier components to permit them to transmit wave forces from the superstructure into the foundation.

Notes:

Pros:

Cons:

Analysis Issues:



TIE PIER CAP TO COLUMNS/PILES

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: