Retrofit Measures for Substructure Components

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Purpose

- To describe typical retrofit measures for:
  - Piers and columns
  - Cap beams
  - Column to cap beam joints
Retrofit Methods

- Column replacement
- Supplemental columns
- Shear & flexure strengthening
- Improvement of ductility
  - Steel jacketing
  - Active confinement w/ prestressing wire
  - Composite fiber/epoxy jacket
  - Reinforced concrete jacket
Column replacement

- Replace when:
  - Damaged column
  - Flexure is greatly insufficient
  - Space limits other measures
Column replacement

- Construction issues
  - Very expensive
  - Longitudinal steel must be anchored into cap
    - Headed reinforcement may be considered
  - Provide pressure grout at top of column

Top of column
Near plastic hinge
Column replacement

- Column anchorage-(headed bars)
Column replacement

- Supplemental column
  - Must ensure existing column continue to carry vertical loads.
Flexural strengthening

- Not desirable in many cases
  - Shear capacity increases which attracts force to the pier cap and foundation
- Desirable case
  - Increase strength at lap splice to drive hinging to new location
- Two common methods:
  - Concrete overlay (additional reinforcement)
  - Adding reinforcement with steel shell
Flexural strengthening

- Concrete overlay
  - Concrete alone adds strength due to moment arm
  - Long steel adds additional strength
  - Must provide shear transfer with dowels
  - Must provide confinement-(hoops)
Column strengthening

- Adding reinforcement with steel shell
Column strengthening

- Adding reinforcement with steel shell
  - Must ensure shear transfer
  - Provide development length into footing/cap
Column strengthening

- Adding reinforcement with steel shell
  - Development of steel shell to ensure plastic strain
Ductility and shear improvement

- Low ductility occurs from using starter bars at base of column
- Concrete splits due to radial stresses from bars trying to pull out
- Proper confinement may stop splice failure
- Extend confinement to 75% of maximum moment or column diameter
Ductility and shear improvement

- Failure path of an unconfined column
  - Spalling begins at strain = 0.005
  - Ties began carrying more stress at this point and fail
  - Confinement is lost and concrete core crushes
  - Vertical steel buckles
  - Conc core fractures and loses axial, flexural and shear capacity
Ductility and shear improvement

- Steel jacketing
  - Use 2 half shells and weld
  - Fill void with grout
  - Confinement is provided AFTER expansion of conc occurs
  - Place 1-2” gap at end of jacket and supporting member
  - R factors=4-6 may be used
  - Stiffness increases 10-15%-Analyze with new stiffness
Ductility and shear improvement
Ductility and shear improvement

- Shell for rectangular columns
  - Rectangular shells do not have membrane action like circular shells. They act like bending members.

![Diagram of reinforced concrete column with steel shell and instructions for clearance and filling with concrete.](image-url)
Steel Shell Retrofit Example (for splice)

- Provide data
- Perform simplified dynamic analysis
- Conduct shell design
- Perform design check
  - Deflection capacity
  - Shear capacity
Structural configuration

Radius = 0.610 m (2.0 ft)

#13 @ 305 mm Hoops
(#4 @ 12"

20 - #35 (#11)

Steel Shell Retrofit

Single Column Pier

9.15 m (30 ft)
Required data (additional)

- Superstructure weight: \( W := 1000 \cdot \text{kip} \)
- Ground acceleration: \( F_{vS1} := 0.6 \cdot g \)
- Concrete strength: \( f' := 5 \cdot \text{ksi} \)
- Effective inertia: \( l_{\text{eff}} := 6.3 \cdot \text{ft}^4 \)
- Nominal moment capacity: \( M_n := 3317 \cdot \text{K ft} \)
- Nominal shear capacity: \( V_N := 110.6 \text{k} \)
- Concrete cover: \( \text{cover} := 2 \cdot \text{in} \)
Simplified dynamic analysis

- Structural stiffness
  \[ k_c := \frac{3 \cdot E \cdot I_{\text{eff}}}{L^3} \]
  \[ k_c = 406.426 \ \text{kN/m} \]

- Mass
  \[ m := \frac{W}{g} \]
  \[ m = 31.056 \ \text{kN/m} \]

- Structural period
  \[ T := 2 \cdot \pi \cdot \sqrt{\frac{m}{k_c}} \]
  \[ T = 1.737 \ \text{s} \]
Simplified dynamic analysis (cont’d)

- Spec acc
  \[ S_a := \frac{FvS1}{T} \]
  \[ S_a = 0.345 \text{ g} \]

- EQ shear
  \[ V_{EQ} := S_a \cdot W \]
  \[ V_{EQ} = 345.454 \text{ kip} \]

- EQ moment
  \[ M_{EQ} := V_{EQ} \cdot L \]
  \[ M_{EQ} = 10363.61 \text{ kip ft} \]

- EQ deflection
  \[ \Delta_{EQ} := \frac{V_{EQ}}{k_c} \]
  \[ \Delta_{EQ} = 0.85 \text{ ft} \]
Shell design

- FBD

Confinement stress

\[ 2 \cdot f_s \cdot t = f_l \cdot D \]

- \( f_s \): stress induced in jacket = 29 ksi at strain = 0.001
- \( f_l \): confinement stress = 300 psi
Shell design

- Shell thickness
  - Use 10mm=3/8 in for handling

- Ductility factor
  \[ t := f_l \cdot \frac{D}{2 \cdot f_s} \]
  \[ R_{\text{demand}} := \frac{M_{\text{EQ}}}{M_n} \]
  \[ t = 0.25 \text{ in} \]
  \[ R_{\text{demand}} = 3.12 \]
  \[ <4 \quad \text{OK} \]

- Plastic hinge zone
  \[ l_p := 0.25 \cdot L \]
  \[ l_p = 7.5 \text{ ft} \]
Design check

- Perform moment curvature analysis to determine rotational capacity
- Conduct pushover analysis to determine rotational demand
- Verify capacity is greater than demand
  - For deflection
  - For shear
Design check

- Perform moment curvature analysis with shell and existing reinforcing
  - Plastic rotation capacity: $\theta_p := 0.0591 \cdot \text{rad}$
  - L x (Øu – Øy)
  - Ultimate strain: $\varepsilon_{\text{cu}} := 0.03211$
  - Deflection at yield: $\Delta_y := 4 \cdot \text{in}$
Design check

- Perform pushover analysis
  - Target displacement from elastic analysis
    \[ \Delta T := 1.5 \cdot \Delta_{EQ} \quad \Delta T = 1.27 \text{ ft} \]
  - Plastic displacement
    \[ \Delta p := \Delta T - \Delta y \quad \Delta p = .94 \text{ ft} \]
  - Plastic rotation demand
    \[ \theta_p := \frac{\Delta p}{L} \quad \theta_p = 0.03 \text{ rad} \]
  - Check rotation demand vs capacity
    \[ \theta_p = 0.03 \text{ rad} < \theta_p := 0.0591 \cdot \text{rad} \]
Design check

- Retrofitted column can handle twice the plastic rotation.
- Actual required thickness would be about 0.16 in but constructability issues for thin shell
Design check

- Shear check outside plastic hinge region
  - Target shear
    \[ V_p := 1.5 \cdot V_N \]
    \[ V_p = 165.9 \text{ kip} \]
  - Shear capacity
    \[ V_c := \phi \cdot A_g \cdot \sqrt{f'_c} \cdot \frac{2.8}{1000} \]
    \[ V_c = 174.02 \text{ kip} \]

OK
Prestress Wrapping

- Better system for confinement (active system)
- Little effect on flexure strength, stiffness
- High losses due to friction
Prestress Wrapping

- Prestress wrapping with wedges
  - Machine pulls strand away from pier and wedges are placed to establish tension
  - Minimum losses
  - No data on rectangular systems
Prestress Wrapping
Prestress Wrapping

- Semi-circular reinforcing bars (same effect as prestress)
Ductility and shear improvement

- Semi-circular reinforcing bars
Ductility and shear improvement

- Composite fiber jacketing
  - High strength glass (E-glass)
  - Carbon
  - Aramid fibers
- Increases ductility/shear and not flexure strength
- Passive system is preferred method
  - (active systems creep and rupture)
- Loses strength due to moisture absorption
- R=4 for most columns
Infill shear wall
Infill shear wall

- Increases shear capacity
- Prevent hinges from forming (transverse)
- Must act composite with existing members (dowels)
Vertical capacity preservation

- Type “P” retrofit
  - Flexure strength not increased due to expansion material
  - Rubble is retained in shell to give vertical support
  - Very cost effective
Limitation of column forces

- Isolation bearings
- Flexural strength reduction
Limitation of column forces

- Sierra Point Overhead
Limitation of column forces

- Isolation
Limitation of column forces

- Not suited for isolation
  - Bridges on soft soil (acc increases)
  - Long period structures (little is gained)
  - Extreme seismicity (large deflections)
Limitation of column forces

- Flexure strength reduction
  - Reduces plastic moment and shear demand
  - Must improve ductility capacity
  - Consider making hinge on frames
Concrete pier wall retrofit

- Stiff and strong in transverse direction
  - Ensure foundation can carry load
  - Retrofit usually not required unless starter bars exist
Concrete pier wall retrofit
Bent cap configurations

- Pinned connections have minimum stress
- Integral caps have significant longitudinal forces

(a) Drop (Separate) Cap

(b) Integral Cap
Bent cap configurations

- Outrigger piers have high torsional stresses due to longitudinal forces
Pier cap/joint replacement

- Consider if column is being replaced
- Difficult
- Expensive
Bent cap strengthening

- Add bolster to side
  - Must act composite to provide flexure and shear
  - Dowels help ensure composite action by shear friction
  - Add extra reinforcing and prestress
Bent cap strengthening

- Integral caps more difficult to retrofit
  - Add additional bars in deck for neg moment
  - Add bolsters to lower side of cap for positive moment
  - Prestressing may be most economical
Reduction in pier cap forces

- Link beam retrofit
  - Creates new critical section below link
  - Limits shear in column to value just below link
  - Must carefully locate beam to ensure cap remains elastic
Strengthening of column and beam joints

- Integral bent cap retrofit
Strengthening of column and beam joints

- Knee joint retrofit
## Pier and column limit states

<table>
<thead>
<tr>
<th>Column or Beam Limit State</th>
<th>Plastic Curvature, $\phi_p^{1,2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression failure, unconfined concrete</td>
<td>$\phi_p = \frac{\varepsilon_{cu}}{c} - \phi_y$</td>
</tr>
<tr>
<td>Compression failure, confined concrete</td>
<td>$\phi_p = \frac{\varepsilon_{cu}}{(c - d')} - \phi_y$</td>
</tr>
<tr>
<td>Buckling of longitudinal bars</td>
<td>$\phi_p = \frac{\varepsilon_b}{(c - d')} - \phi_y$</td>
</tr>
<tr>
<td>Fracture of longitudinal reinforcement</td>
<td>$\phi_p = \frac{\varepsilon_{s,\text{max}}}{(d - c)} - \phi_y$</td>
</tr>
<tr>
<td>Low-cycle fatigue of longitudinal reinforcement</td>
<td>$\phi_p = \frac{2\varepsilon_{ap}}{(d - d')} = \frac{2\varepsilon_{ap}}{D'}$</td>
</tr>
<tr>
<td>Lap-splice failure: (a) long / confined lap-splices</td>
<td>See low cycle fatigue \phi_p = (\mu_{lap} + 7)\phi_y</td>
</tr>
<tr>
<td>(b) short / unconfined lap-splices</td>
<td></td>
</tr>
<tr>
<td>Shear failure: (a) brittle</td>
<td>$\phi_p = 0$</td>
</tr>
<tr>
<td>(b) semi-ductile</td>
<td>$\phi_p = \left(5\frac{V_m - V_f}{V_i - V_f} + 2\right)\phi_y$</td>
</tr>
<tr>
<td>Joint or connection failure: (a) weak joint / strong column</td>
<td>$\theta_p = 0.04 \text{ rad}$</td>
</tr>
<tr>
<td>(b) semi-ductile</td>
<td>$\phi_p = \left(4\frac{V_{bh} - V_{hf}}{V_{bf} - V_{hf}} + 2\right)\phi_y$</td>
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**Notes:**
1. Hinge rotation $\theta_p = \phi_p L_p$ where $L_p$ is length of plastic hinge
2. Notation is defined in section 7.8.2.
Pier and column limit states

Moment-Curvature

Moment

Curvature

Force

Displacement

C.L. Column

C.G.

Idealized Yield Curvature

Equivalen Curvature

Actual Curvature

\( \phi_u \)

\( \phi_p \)

\( \phi_Y \)

\( \Delta_c \)

\( \Delta_p \)

\( \Delta_{y}^{col} \)

\( \Delta_y \)

\( \Delta_u \)

\( \theta_p \)

\( L \)

\( L_p \)
Summary

- To describe typical retrofit measures for:
  - Piers and columns
  - Cap beams
  - Column to cap beam joints
What questions do you have?