UNSEATING OF DETERIORATED CONCRETE BRIDGES

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Abstract— Span unseating is the most common failure mechanism during seismic activities. Ground shaking and deformations during an earthquake can cause severe translation in the superstructure causing unseating in the supporting span. High levels of corrosion in bridge seats cause concrete to crack and spall, which results in decreasing the support length. This can result in a catastrophic collapse of the bridge span during an earthquake. This study proposes a practical, safe, and cost-effective solution for the rehabilitation of deteriorated bridge bents to avoid the unseating failure mechanism.

Index Terms - Unseating, Bridge, Seismic, Deterioration, Rehabilitation.

I. INTRODUCTION

Unseating occurs in the superstructure and is usually a result of the large displacements encountered during an earthquake and it is the most common failure mechanism (Saad et al. 2010). The unseating and collapse of bridge spans is due to excessive longitudinal movement at in-span hinges, supports, or expansion joints.

Span Collapse of bridges from around the world during past earthquakes has shown the vulnerability of bridges to this mode of collapse. Bridges have collapsed due to unseating in many earthquakes such as the Niigata earthquake in 1964 (Japan), San Fernando Earthquake in 1971, Loma Prieta earthquake in 1989, Northridge earthquake in 1994, Kobe earthquake in 1995 (Japan), Chi-Chi earthquake in 1999 (Taiwan), and Kocaeli Earthquake in 1999 (Turkey) (Fig. 1).

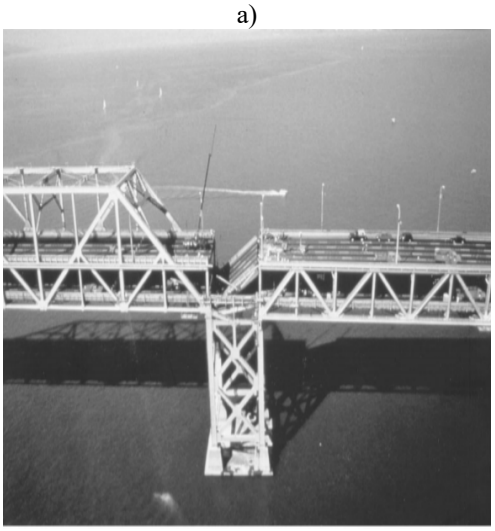
Unseating of a span usually occurs due an earthquake because of inadequate support length. When the relative horizontal displacement at the deck exceeds the support at the intermediate hinge or abutment, the girder can deck can unseat and collapse. This is because many bridges are designed according to older seismic codes and do not provide adequate seat width.

Even though that many of older bridges have been retrofitted using hinge restrains, some bridges during the 1989 Loma Prieta earthquake experienced several cases of hinge restrainer failures (Saiidi et al. 1993). Moreover, the 1994 Northridge earthquake caused many restrained bridges to collapse due to unseating (Moehle 1995).

However, deterioration of the concrete bridge pier caps can lead to a shortening in the support length, which, in turn, leads to a catastrophic collapse of the bridge span. In addition, concrete deterioration and spalling due to corrosion of steel reinforcement at the end of girders can result in unseating of bridge deck.



a)



b)

Fig. 1: a) Showa Bridge Collapse, b) San Francisco–Oakland Bay Bridge Collapse (Moehle, 2000)

II. MINIMUM SUPPORT LENGTH REQUIREMENTS

AASHTO requires a minimum support length at expansion bearings without restrainers, Shock Transmission Units, or dampers to be a percent of the of the empirical support length, N, specified by Eq. 1.

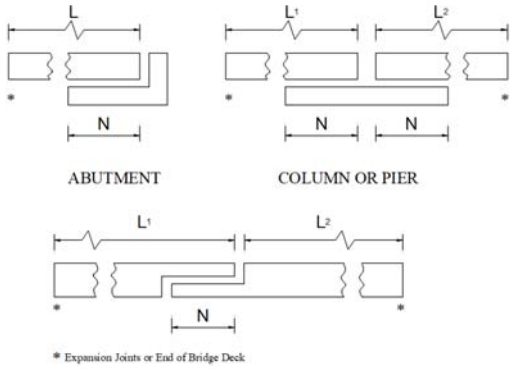


Fig. 2: Support Length, N, AASHTO

$N = (8 + 0.02L + 0.08H)(1+0.000125S^2)$ Eq. 1

where:

N = minimum support length measured normal to the centerline of bearing (in.) (Fig. 2)

L = length of bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck

H = for abutments, average height of columns supporting the bridge deck from the abutment to the next expansion joint (ft); for columns and/or piers, column, or pier height (ft); for hinges within a span, average height of the adjacent two columns or piers (ft); for single-span bridges, 0.0 (ft)

S = angle of skew of support measured from a line normal to span (^{o})

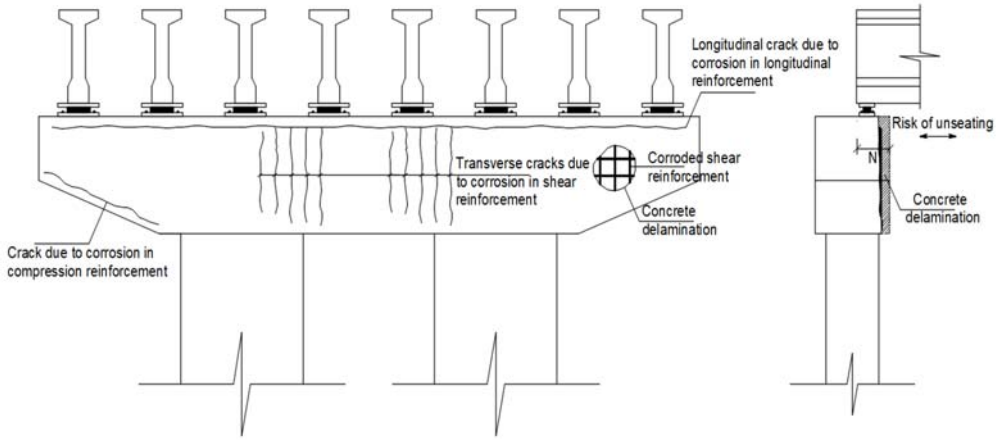


Fig. 3. Deterioration of Bridge Bent Increase the Risk of Unseating During an Earthquake

Table 1: Percentage N by Zone and Acceleration Coefficient A_S

Zone	Acceleration Coefficient, AS	Percent, N
1	<0.05	≥75
1	≥0.05	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

Minimum support length provisions provided above are equivalent to the AASHTO LRFD Bridge Design Specifications Article 4.7.4.4.

Support lengths are equal to the length of the overlap between the girder and the seat as shown in Fig. 2. The minimum values for N given in the above equation include an arbitrary allowance for cover concrete at the end of the girder and face of the seat. If above average cover is used at these locations, N should be increased accordingly (AASHTO Guide Specifications for LRFD Seismic Bridge Design).

III. DETERIORATION OF SEATS AND CAPS

While some bridges provide drainage systems to collect debris and deicing salt that falls through the expansion joints, most of the bridges lack these systems. The accumulation of debris, water, and deicing salt on the substructure caps over time will cause water and deicing salt to penetrate into concrete. The presence of salt water on the steel reinforcement will tremendously accelerate corrosion. As steel corrodes, it expands because of the greater volume of the corrosion byproducts before it starts losing its volume. The expansion of steel reinforcement due to corrosion causes cracking, spalling, and delamination in the concrete cover decreasing the required seating area discussed above as shown in Fig. 3 and Fig. 4. Even when bridges are equipped with drainage systems, corrosion can still occur due to clogging and run-over. Moreover, corrosion of anchorage bolts causes extensive cracking in surrounding concrete, which also leads to spalling of concrete.

Delamination of concrete will decrease the minimum support length required by AASHTO, which in turn, will increase the risk of span collapse during an earthquake due to unseating. Moreover, when concrete starts to crack, spall, and delaminate due to the radial stress caused by corroded reinforcing steel, it will increase the access of water and deicing salt to the non-corroded steel reinforcement.

This will accelerate the corrosion process in the non-corroded bars, leading to an increase in delamination of seat concrete and a higher risk of unseating.



Fig. 4. Deterioration of Bent Cap (http://www.modot.org)

IV. REHABILITATION SCHEME

Corrosion of steel bars compounds the deficiencies of concrete bent caps. The first step for seismic retrofit of deteriorated concrete bridge components is the restoration of the original condition of the bridge. Lost steel sections and spalled concrete should be replaced, and the bond between the steel rebars and the surrounding concrete should be restored. Restoration to the original condition should be associated with adoption of corrosion protection system.

The following steps are recommended for restoring the original condition of the cap

1. First, the bridge shall be closed for traffic and the road under the bridge shall also be closed. High Strength Bridge Scaffolding might be required to support and stabilize the superstructure during the repair process.
2. Removal of the damaged concrete. Deteriorated Concrete can be removed manually, or any other concrete removal technique such as hydroblasting, bush hammering, or concrete saw. No deteriorated concrete shall remain adjacent to the corroded reinforcement.

   Concrete must be removed completely around the corroded rebars allowing for a minimum space of 3/4 inch (19 mm) clearance on all sides. Concrete edges must be cut at 90º and featheredging shall be avoided as shown in Fig. 5. The 90º edges are important to provide a better bond between old and new concrete, deliver a retaining boundary, and lock the repairing concrete into the repaired area, especially when the deterioration does not extend over the full length of the cap.
3. Cleaning the concrete surface. As a result of the removal of deteriorated concrete, the interconnected microcracks in the concrete substrate will create a weak surface layer. This layer will reduce the bond strength if left in place as it can extend to a depth of 1/8 to 3/8 in. (3 to 10 mm) into the resultant concrete substrate. High-pressure water blasting or oil-free abrasive blasting is recommended to remove the weakened layer and other bond inhibiting materials such as dirt and loose concrete (Fig. 6). A pressure of 8,000 to 15,000 psi is effective in removing concrete in low quality while keeping sound concrete in place.

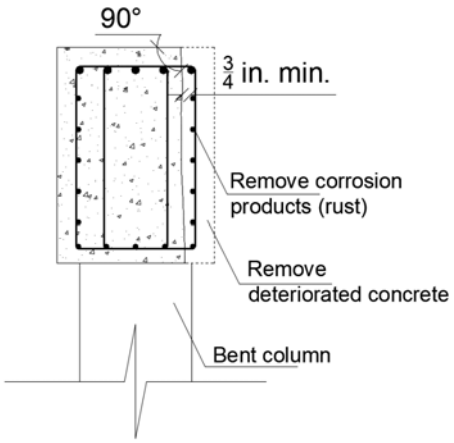


Fig. 5. Removal of Deteriorated Concrete and Cleaning Steel Rebars

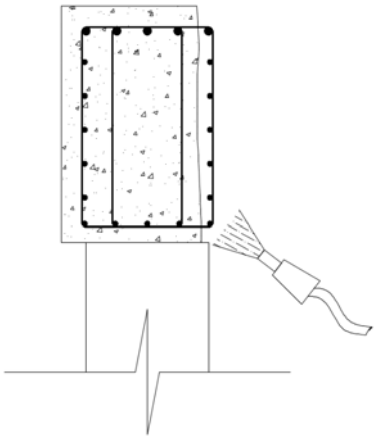


Fig. 6. Cleaning Concrete Surface with High-Pressure Water Blasting

4. Cleaning steel rebars and removing any corrosion product. It is extremely important not to damage the bond between steel rebars and the surrounding concrete if no corrosion is present at the steel reinforcing bars. All rust, concrete, and scale must be removed from the surface of the steel reinforcement.
5. If loss of steel cross-sectional area exceeds 25%, steel rebar should be replaced or supplemental steel rebars should be added in the damaged in

accordance with section 12.14 of American Concrete Institute (ACI) 318 (ACI, 1992). The supplemental rebars should be mechanically spliced or lap-spliced to the existing rebars.

Table 2) as follows:

Class A splice……... 1.0 ℓ_d

a) 1. AASHTO (5.11.5.3) requires the length of lap for tension lap splices not be less than either 12.0 in. or the following for Class A, B or C splices (

Class B splice…… 1.3 ℓ_d
Class C splice…… 1.7 ℓ_d

Table 2. Classes of Tension Lap Splices (AASHTO Table 5.11.5.3.1-1)

Ratio of $\frac{(A_s \text{ as provided})}{(A_s \text{ as required})}$	Percent of A_s Spliced with Required Lap Length		
	50	75	100
≥2	A	A	B
<2	B	C	C

The tension development length, ℓ_d shall not be less than the product of the basic tension development length, ℓ_{bd}, specified herein and the modification factor or factors specified in Articles 5.11.2.1.2 and 5.11.2.1.3. The tension development length shall not be less than 12.0 in., except for lap splices specified in Article 5.11.5.3.1 and development of shear reinforcement specified in Article 5.11.2.6.

The basic tension development length, ℓ_{bd}, in in. shall be taken as:

For No. 11 bar and smaller .... $\frac{1.25 A_b f_y}{\sqrt{f_c'}}$

but not less than .... $0.4 d_b f_y$

For No. 14 bars .... $\frac{2.7 f_y}{\sqrt{f_c'}}$

For No. 18 bars .... $\frac{3.5 f_y}{\sqrt{f_c'}}$

For deformed wire .... $\frac{0.95 d_b f_y}{\sqrt{f_c'}}$

where :

A_b = area of bar or wire (in. ^{2})
f_y = specified yield strength of reinforcin g bars (ksi)
f_c' = specified compressiv e strength of concrete at 28 days, unless another age is specified (ksi)
d_b = diameter of bar or wire (in.)

AASHTO recommends lap splices not be used for bars larger than No. 11 for reinforcement in tension. In addition, bars spliced by noncontact lap splices in flexural members shall not be spaced farther apart transversely than one-fifth the required lap splice length or 6.0 in. The length of lap, ℓc, for compression lap splices, recommended by AASHTO (Article 5.11.5.5.1) shall not be less than 12.0 in. or as follows:

If $f_y \le 60.0$ ksi then:

$\ell_c = 0.5 m f_y d_b$

or

If $f_y > 60.0$ ksi then:

$\ell_c = m(0.9 f_y - 24.0) d_b$

in which:

Where the specified concrete strength f_c', is less than 3.0 ksi..... $m = 1.33$
Where ties along the splice have an effective area not less than 0.15 percent of the product of the thickness of the compression component times the tie spacing..... $m = 0.83$
With spirals..... $m = 0.75$
In all other cases..... $m = 1.0$

where

f_y = specified yield strength of reinforcing bars (ksi)
d_b = diameter of bar (in.)

6. Set the dowels for formwork support and install the forms around the existing concrete member. Apply epoxy-bonding agent to all concrete surfaces that will be in contact with new repairing concrete before pouring the new concrete. The new concrete should have the same concrete mix as the concrete in place in order to provide reliable repair. Allow concrete to cure and develop adequate strength to resist the loads before removing the formwork.
7. Install corrosion prevention system, such as surface-applied liquid chemicals or sacrificial anodes to avoid future corrosion in the steel rebars. A surface-applied liquid chemical is self-penetrating corrosion inhibitor that can be applied to the concrete surface using a roller or a spray. The corrosion inhibitor protects the steel rebars by penetrating into concrete and forming a protective layer on the steel surface. Sacrificial anodes (SA) are extremely active metals that are used to prevent a less active material surface from corroding. they consist of an active cementitious matrix surrounding a galvanic zinc core. At selected locations, pre-attached tie wires are used to attach the SAs directly to the steel reinforcing bars. They provide localized corrosion protection in reinforced concrete bridges. By integrating these small, zinc-based anodes into concrete repairs, concrete durability can be extended by 10-15 years. Some of the main advantages of the sacrificial anodes are:

a) Very active in preventing corrosion
b) Advantageous at locations where chloride contaminated concrete cannot be removed
c) Very effective regardless of the condition of the surrounding concrete because they target the steel rebars directly
d) Provide long service life

8. It is highly recommended to use Fibre Reinforcement Polymer (FRP) composites as a strengthening system after restoring the original shape of the cap. Not only do FRP composites increase the cap ultimate strength and stiffness, but they also provide crack control for the concrete cap. Moreover, they provide confinement, which

increases the compressive strength of the concrete member (Fig. 7).

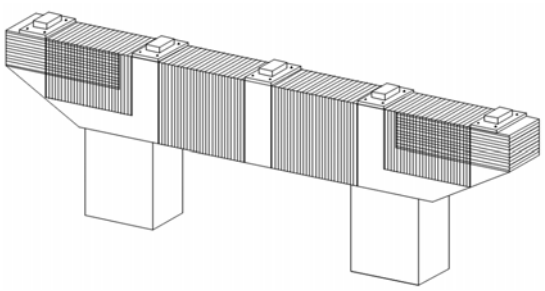


Fig. 7. Bridge Pier Strengthened with FRP Composites

9. Note that if the seat width before deterioration does not meet the minimum support width required by AASHTO, the width of the bent cap must be increased during the rehabilitation to meet the minimum support length provided in Eq. 1.

SUMMARY AND CONCLUSIONS

Unseating is the most common earthquake failure mechanism. This mode of failure occurs during an earthquake as a result of inadequate support length. High levels of corrosion in bridge bents cause spalling and delamination in the concrete cap, which, in turn, results in decreasing the support length resulting in unseating of girders/deck during seismic activities. This study discusses the consequences of unseating failure and suggests practical, safe, and cost-effective retrofit procedure to restore the required seating length of corroded pedestal over piers and abutments.

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