Damage to Grier St. POC [Br. #53-1158]
Post-Earthquake Investigation Team Report

Acknowledgments

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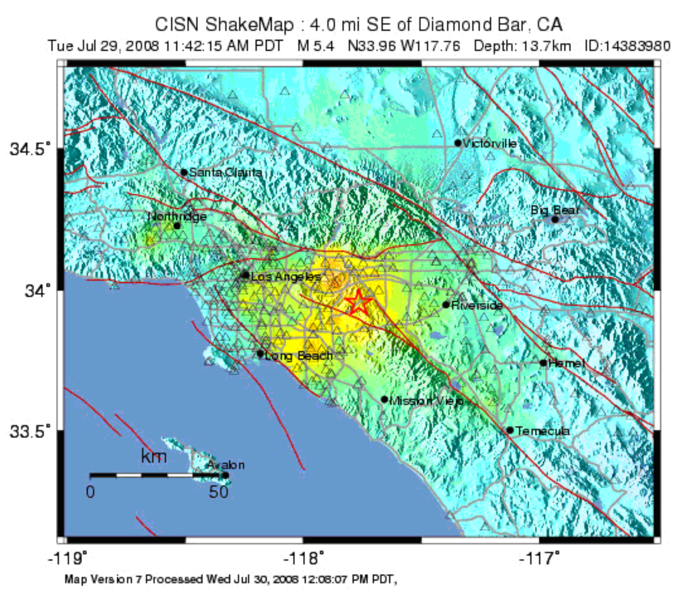
Seismological Data

Late in the morning on July 29th, 2008, a M5.4 earthquake shook the greater Los Angeles area. The earthquake was the strongest to hit southern California since the 1994 Northridge earthquake and occurred at a depth of about 9 miles. The aftershock sequence from the earthquake has been about 75 percent less energetic than the average California aftershock sequence for a M5.4 mainshock, probably because this event was deeper than some southern California earthquakes.

The event was felt across southern California, with strong shaking reported to the north in the Chino Basin and to the west in the Los Angles basin (Figure 1) California State University, Fullerton, located about 10 miles west suffered some damage in its older buildings. A minor landslide near Route 91 in the Anaheim Hills 7 miles to the south caused some traffic congestion, but no injuries or structural damage were reported.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 1 Shake Map.

Ground shaking predicted by the latest ground motion prediction equations developed for use in the 2008 USGS seismic hazard maps fits well with most observed recordings of this event. The greatest acceleration observed (horizontal PGA) was about 0.44 g recorded near Walnut, about 11 km northwest of the epicenter. Peak velocity measured at the Walnut station was 38 cm/sec and the duration was a few seconds. Recordings in the greater Los Angeles area had peak accelerations from 0.10g to 0.20g and peak velocities were from 5 to 15 cm/sec. There were very few reports of damage. According to Sue Hough of the USGS, the earthquake occurred in a complicated tectonic region — the wedge sandwiched between the Whittier and Chino Hills faults (Figure 2 that probably has a lot of small faults with different orientations. The current theory is that the wedge described above conceals a structure known as the Yorba Linda trend with no surface expression but identified as a linear trend of seismicity by Caltech seismologist Egill Hauksson in 1990. The moment tensor for this earthquake showed a mixture of thrust and left-lateral strike-slip faulting on a plane striking 43 deg east of north, with has a dip of 58 deg southerly. The preliminary locations of the aftershocks suggest that this is the fault plane. Updates to this information may be found at:

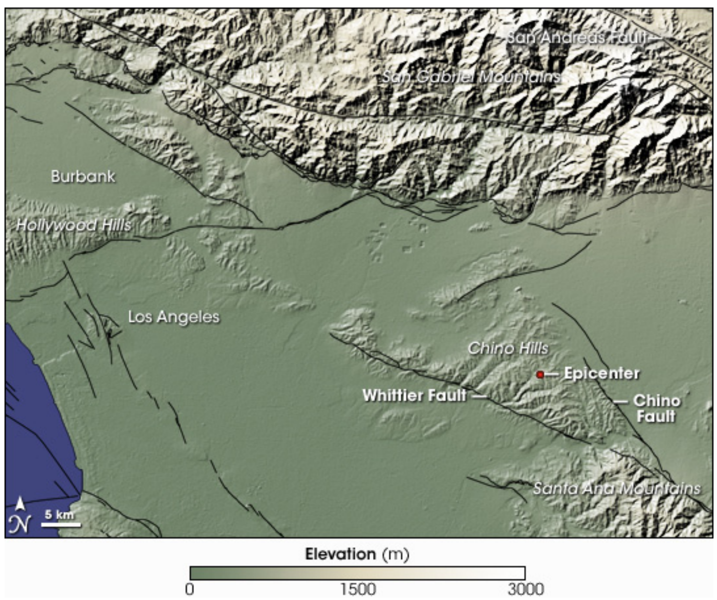


Figure 2 Location of the epicenter of the July 29, 2008 M5.4 Chino Hills earthquake.

Bridge Description

The Grier St. POC is located at Post Mile 1.2 on Route 71 in Los Angeles County in the city of Pomona. This bridge is a simply supported eight span precast prestressed voided slab pedestrian overcrossing bridge. The precast slabs (two 4 ft. wide slabs placed side by side) are supported on rectangular reinforced concrete pier bents and seat type abutments. The top of the pier is only two feet wide, and this seat is shared by adjacent spans, making the seat width slightly less than one foot (Figure 3) The two precast slabs are doweled and grouted to the piers with 2 - # 8 bars each (total of four grouted dowels at each end of a span) at both supporting piers (Pier 4 and Pier 5). The precast slabs are overlain with AC. Originally constructed in 1958, the bridge was retrofitted in 1998 by enlarging the footings and adding CIDH piles at Piers 4, 5, and 6 (see Attachment 1).

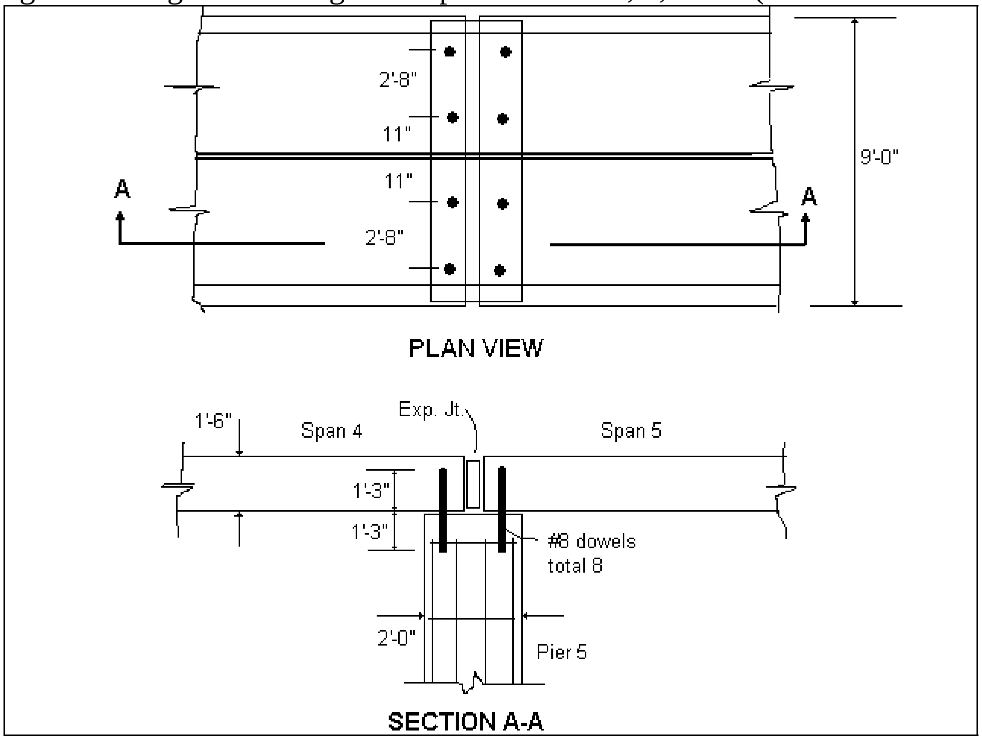


Figure 3 Support Connection at Pier 5.

Bridge History

The Structural Maintenance and Investigations (SM&I) Bridge Inspection Report (BIR) recorded the following bridge description and history:

A. BIR 8/23/2000:
   Records showed the structure appears to be in good condition.
B. BIR 8/13/2002:
   Records revealed a spall due to an over-height hit over the NB lane (Span 5) and a similar hit spall over the SB lane (Span 4). The spall at Span 4 appears to be larger than that at Span 5 (200%). Also, it was noted that the bridge was proposed to be removed under project EA 07-21060 in fiscal year 2005.
C. BIR 8/20/2004:
   Records showed the structure appears to be in good condition.
D. BIR 8/16/2007:
   Records showed numerous over-height hit spalls over the NB lane (Span 5). The report did not mention the three pocket spalls on Span 4 (SB lane), as shown in Figure 4 (taken on 8/4/2008). In addition, the bridge has been identified as one of the bridges requiring work to comply with the Americans with Disabilities Act (ADA) requirements. Note that the earlier contract to widen the northern segment of Route 71 wherein this POC was to be removed (EA 07-21060) was never completed, and thus the POC remains in service.

Figure 4 Damage to superstructure from over-height hits in Span 4

Damage Observations

Spans 4 and 5 are the main spans crossing Route 71 over the southbound and northbound directions. Based on the site visit on 8/4/2008 (after the M5.4 Chino Hills earthquake), visible structural damage has been observed at the connections between Span 4 and Piers 4 and 5.

Pier 5 Damage

Concrete spalled at both ends of the top of Pier 5 supporting the Span 4 slab, exposing one dowel at each side. The two exterior dowels at each end of the are bent and the Span 4 slab exhibited a permanent transverse displacement of about 3 to 4 inches (toward the Southeast direction), as shown in Figure 5.

The exposed spalled area of concrete at the Northwest end of the Pier 5 seat exhibits rusty yellow stains (Figure 6). The exposed stains indicate that the area was cracked prior to the Chino Hills Earthquake. In addition, rust was observed on the two exterior dowels, which are now bent. The previous concrete cracking at the top edges of the Pier 5 may be related to the truck over-height hits on the Span 4 slab.

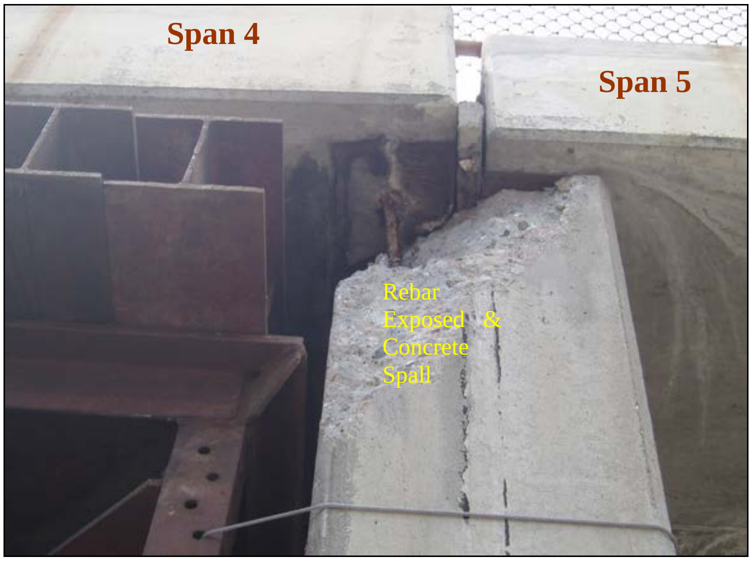


Figure 5 Concrete spalls at the southwest corner of Pier 5; note the evidence of corrosion on the dowel connecting the superstructure voided slab to the pier

Figure 6 Concrete spalls and stains at the northwest corner of Pier 5 (note stains).

Pier 4 Damage

Concrete spalls and cracks were observed at the top the piers supporting the Span 4 slab. Small pieces of concrete (triangular shapes) had fallen off and cracks were visible at the exterior face of Pier 4. Similar to Pier 5, the slab in Span 4 had a permanent offset, displacing transversely (toward the Southeast direction) at Pier 4.

The ground motion pushed the Span 4 slab transversely, bent the dowels, and induced transverse displacement on Span 4 (Figure 7). The major observed failures were bending of the dowel bars connecting the super structure slab with the pier, and concrete cracking and spalling at the corners of the piers supporting Span 4.

In addition, no damage was observed at the connections between Span 5 and the supporting piers, even though Span 4 and Span 5 share the same design and connection details.

Figure 7 Concrete cracking and spalling at Pier 4 under Span 4.

Damage Assessment

- The dowels may not be strong enough to resist the transverse displacement.

- The transverse reinforcement for the piers, as shown in the plan and elevation details for Pier 5 on the as-built plans, are #4 ties @ 12" with the top one foot of the pier apparently not reinforced transversely (unconfined). This is substantiated by the fact that there is no evidence of transverse reinforcement in the cracked sections near the top of the pier. The superstructure to pier connection dowels are embedded into this top 1 foot of unconfined concrete. The lack of confinement allowed cracking and spalling of the top of the pier and the subsequent reduction in the dowel shear capacity.

- It is theorized that previous over-height truck hits on Span 4 may have induced cracks at the top of Pier 5. The ground motions from the Chino Hills Earthquake on 7/30/2008 placed further demands on the weakened connection, spalling the concrete and resulting in the bending of the connection dowels.

Actions by Structures Maintenance and Investigations

The extent of transverse displacement at the piers does not seem to affect the overall structural stability, and no imminent collapse is anticipated at this time. The structure is currently closed. In addition, Span 4 is supported with temporary falsework at Pier 5.

Lessons Learned

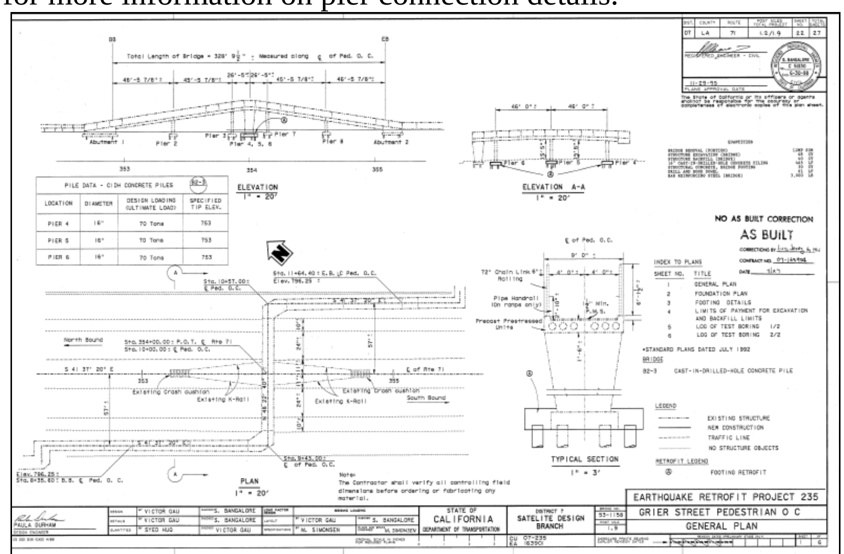
The investigators believe the damage noted in Span 4 resulted from the combination of previous over-height hits and the Chino Hills Earthquake. Analysis of the connection reveals that the D/C ratio is approximately 0.5 for the undisturbed condition, and thus the earthquake was not solely responsible for the damage. This is further substantiated by the fact that only Span 4 and the portions of Piers 4 and 5 supporting Span 4 exhibited damage from the earthquake. Analysis considering pre-existing damage to the dowels connecting the superstructure to the piers at the edges where rust staining indicates previous damage supports the above as the capacity is much reduced.

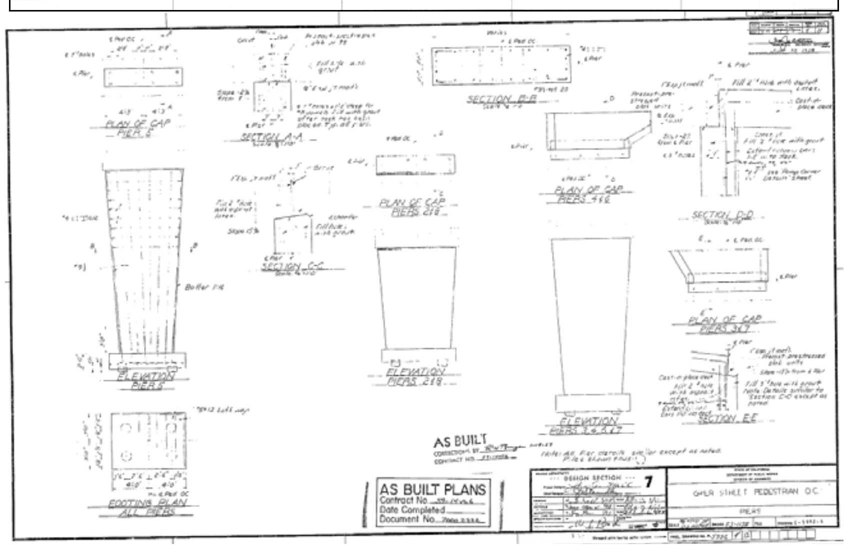
Still, bridges with poor confinement details at the top of the pier and piers with exterior dowels that have a small concrete cover should be considered for retrofit. Earthquakes that produce higher ground motions or more cycles of motion could cause these connections to fail and the girders to fall from their support.

ATTACHMENTS

Attachment 1: As Built Plans

See the general plan for the bridge geometry and dimensions. See the as-built pier detail sheet for more information on pier connection details.





Attachment 2: Seismic Evaluation of As-Built Dowel Connection

The following calculation shows the capacity of #8 dowel bars connecting superstructure to the piers for lateral shear forces and the estimated demand forces exerted on the joint generated by Chino Hill M5.4 earthquake.

Capacity Analysis (Transverse Direction)

Capacity of Dowel Bar to Lateral Forces

1.) Assume the Bridge is in as built condition

Check the Capacity of the dowels to resist lateral Force

Assumption: Concrete has aged enough to get ultimate expected strength of
$f'c = 5$ Ksi
$Ec = 4200$ Ksi
Unit Wt = 0.150 Kip/ft^3

Steel have expected yielding strength
fye = 68 Ksi , fy = 60 [for shear]

Assume there are no cracks in the concrete dowel connections

There are four #8 bars each embedded 1.25 feet into the bridge pier.

$l_d = 1.25' = 15''$
$A_{vp} = 0.79\ in^2$
$d_b = 1$ in 5"

Required Development Length for # 8 Bar in Tension

$l_{db} = 1.25 * A_{cv} * fy * /(f'c)^{0.5} = 1.25*0.79*60/(5)\text{^}0.5 = 29.97$ in
$l'_d = 0.8 * 29.97 = 23.9$, Use 24in

Factor to be used to correct Shear Capacity, $f = l_d / l'_d = 15''/24'' = 0.625$

Shear Friction Capacity of single dowel bar

$Vn = C * A_{cv} + \mu *(A_{vp} *fy + Pc)$ ---------- ---------→ 5.8.4.1 (AASHTO LRFD)

Vn - Nominal shear capacity of single dowel bar
μ - = 0.6 -→ [for contact between #8 bar and pre-cast slab]
C - =0.075 Ksi Use 0.0 Seismic →[cohesion between concrete and bar]
Pc - = 0.0 → [No compressive force]
A_{cv} - Area of concrete engaged with embedded #8 Bar

$A_{cv} = \Pi * d_b * l_d = 3.1415 * 1 * 15 = 47.1\ in^2$

$Vn = C * A_{cv} + \mu * (Avp * fy + Pc)$

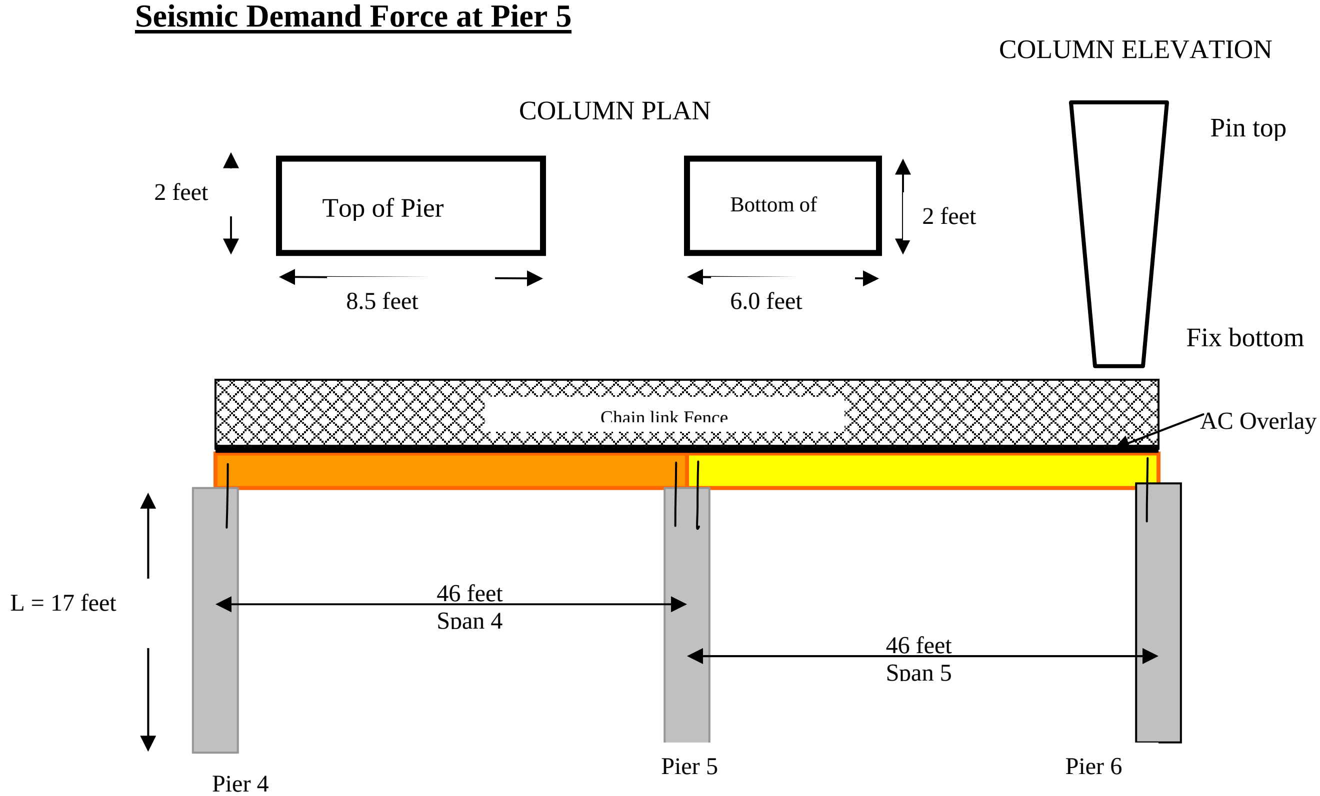
$f * Vn = 0.625 * [0.0 * 47.1 + 0.6 * (0.79 * 60 + 0)] = 17.775$ Kip

There are four #8 Bars per pier per span

$\Sigma Vn = 4 * 17.775 =$ **71.1 Kips** at each end of Span / Pier Connection.

Demand Analysis (Transverse Direction)

Seismic Demand Force at Pier 5



$I_{strong} = Bh3 / 12 = (2*8.5^3)/12 = 102.3\ ft^4$ -------→ Top of Pier

$I_{strong} = Bh3 / 12 = (2*6.0^3)/12 = 36.0\ ft^4$ -------→ Bottom of Pier

Take the Average for simplification of calculation

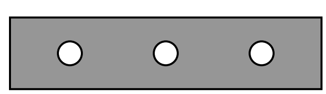
$I_{strong} = 70\ ft^4$

$E = 4200$ ksi

Super Structure

Pre-Cast Void Slab

Area = $673.2\ in^2$



4' Wide 1.5' Deep Void
Pre-Cast Slab

W1 (Weight of Pre-Cast slab per foot length of each Span)
W2 AC overlay per foot length (1" AC overlay = 0.05 ton/Sq-yd)
W3 (Chain Link Fence) per foot length = 0.12kip/ft

W1 = 673.2 /144 * 0.150*2 = 1.40 Kip/ft -------→ Two slabs side by side
W2 = 0.05*8/9 *(2 Kip/Ton) = 0.09 Kip/ft
W3 = 0.12 *2 =0.24 Kip/Ft -------→ Two chain link fence on each side

$m = \Sigma W = 1.73$ kip/ft

Take Pier5

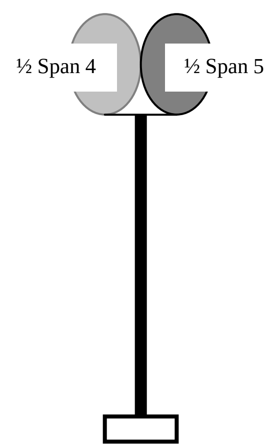
Local Analysis

Contributory span length of Pier5 is 23feet from span 4 and 23feet from Span5

$\mathbf{m} = 46 * 1.73 = 79.6$ Kip

$K = 3 E * I * \mathbf{g} / L^3$

$K = (3 * 4200 * 70 * 12^4) * 386.4 / (17*12)^3 = 832415.5$ [Kip/in in/Sec^2]



Pier 5

Determine Fundamental Period of Frame in transverse direction

$$\mathbf{T = 2 * Л * (m/K)^{0.5}}$$

$T = 2 * 3.1415 * (79.6/832415.5)^{0.5} = 0.061$ sec

Compare Sap 2000 Period T = 0.075 Sec

Using Upper Bound Response Spectrum Data [conservatively]

Use T = 0.07sec
Associated Spectral Acceleration
$a = 0.8 * \mathbf{g} = 0.8*32.2 = 25.75$ ft/ sec^2

Total Force at the joint by Span 4 on Pier 5

F= 23[ft] * 1.73[Kip/ft] * 0.8 * $\mathbf{g}$ [ft/ sec^2] = **<u>31.8 Kip</u>**

Demand Force on Four #8 Bar in [Kip] = 23*1.73*0.8 = <u>31.8 Kip</u>

Capacity of Four # 8 Bar in [Kip] = <u>71.7 Kip</u>

Capacity / Demand ≈ 2

Conclusion:

As demonstrated by the adjacent undamaged spans, Span 4 would have sustained the Chino Hills EQ seismic forces, if prior damage had not occurred at the connections (dowels) due to over-height truck hits and due to the poor confinement detail.