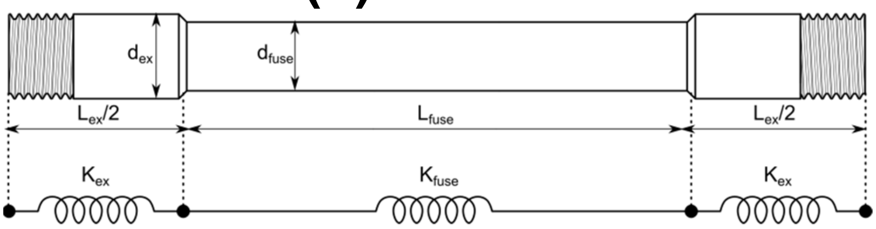
5d.(ii) Mild steel



Yield displacement

$$\Delta_y = \frac{k_a f_y L_{fuse}}{E}$$

$$k_a = \left(\left(\frac{d_{fuse}}{d_{ex}} \right)^2 \frac{L_{ex}}{L_{fuse}} + 1 \right)$$

Post-Yield displacement

$$\Delta = \Delta_y + \frac{\left(f_s - f_y\right) L_{fuse}}{E} \frac{1}{r}$$

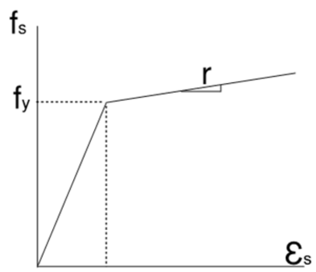
5d.(ii) Mild steel

- A bi-linear stress strain response is adopted for the reinforcing bars with the bar stress given by:

$$f_s(i) = \varepsilon_s(i) E_s \quad if\, \varepsilon_s(i) < \varepsilon_y$$

$$f_s(i) = f_y \left(1 + r \left(\frac{\varepsilon_s(i)}{\varepsilon_y} - 1 \right) \right) \quad if\, \varepsilon_s(i) > \varepsilon_y$$

- Where r is the bi-linear factor which is commonly taken as 0.8% for mild steel dissipaters.

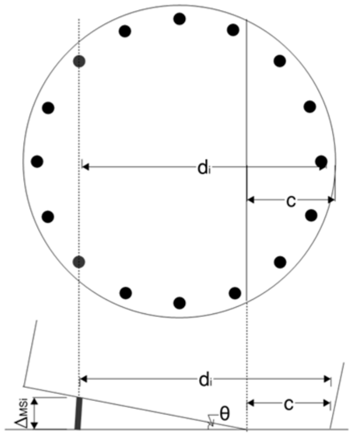


5d.(ii) Mild steel

Summary of mild steel bar forces

$Layer, i$	$d(i)$	$n_b(i)$	$A_s(i)$	$\Delta_s(i)$	$\varepsilon_s(i)$	$f_s(i)$	$T_s(i)$
	mm		mm²	mm	με	MPa	kN
1	70	1	881	-15.8	-20285	-578	-510
2	76	2	1763	-15.7	-20075	-578	-1018
3	95	2	1763	-15.1	-19410	-577	-1017
4	125	2	1763	-14.3	-18361	-575	-1014
5	167	2	1763	-13.2	-16892	-573	-1009
6	220	2	1763	-11.7	-15039	-570	-1004
7	284	2	1763	-9.98	-12800	-566	-998
8	357	2	1763	-7.99	-10247	-562	-991
9	439	2	1763	-5.76	-7379	-557	-983
10	529	2	1763	-3.30	-4232	-552	-974
11	625	2	1763	-0.68	-874	-175	-308
12	726	2	1763	2.07	2658	532	937
13	832	2	1763	4.96	6365	556	980
14	940	2	1763	7.91	10142	562	990
15	1050	2	1763	10.9	13989	568	1001
16	1160	2	1763	13.9	17837	574	1012
17	1268	2	1763	16.9	21614	580	1023
18	1374	2	1763	19.8	25321	586	1033
19	1475	2	1763	22.5	28853	592	1043
20	1571	2	1763	25.1	32211	597	1053
21	1661	2	1763	27.6	35358	602	1062
22	1743	2	1763	29.8	38226	607	1070
23	1816	2	1763	31.8	40779	611	1077
24	1880	2	1763	33.6	43018	614	1083
25	1933	2	1763	35.0	44871	617	1088
26	1975	2	1763	36.1	46340	620	1093
27	2005	2	1763	37.0	47389	621	1095
28	2024	2	1763	37.5	48054	622	1097
29	2030	1	881	37.6	48264	623	549
						$\Sigma T_s(i)$	8460

Check <5%



5d.(iii) Concrete stress block

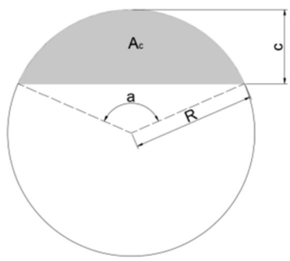
- The concrete compression force is usually calculated from:

$$C_c = \alpha f_c' \beta c B$$

where B is the section width.

- However for circular column, the concrete compression area cB is replaced with $90\% A_c$ as described in Eurocode 2 Cl 3.1.7(3) giving:

$$C_c = 0.90 \alpha f_c' \beta A_c$$



$$a = 2cos^{-1}(1 - \frac{c}{R})$$

$$A_c = \frac{R^2}{2}(a - sina)$$

5d.(iii) Concrete stress block

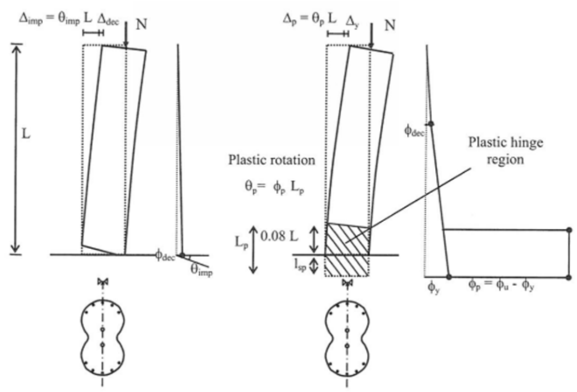
$$C_c = 0.90\alpha f_c' \beta A_c$$

- Convention assumption of section strain compatibility doesn't apply.
- We can't use regular α and β values from NZS3101. We need to get values which are derived from the Monolithic Beam Analogy (MBA).
- Equivalent monolithic section is used to calculate the concrete strains in the non-emulative connection.

5a. Moment and gap 5b. PT Quick 5c. MS Quick **5d. Detailed design** 5e. Cyclic 5f. Detailing

5d.(iii) Concrete stress block

Modified Monolithic beam analogy (Palermo, 2004)



$$\Delta_{rocking} = \Delta_{monolithic}$$

5d.(iii) Concrete stress block

$$\varepsilon_c = \left[\frac{\theta L_{cant}}{\left(L_{cant} - \frac{L_p}{2}\right) L_p} + \phi_y\right] c$$

- Cantilever length:

$$L_{cant} = H = 7000\ mm$$

- Yield curvature:

$$\phi_y = \frac{2.25\varepsilon_y}{D} = 2.95 \times 10^{-3}\ m^{-1}$$

Section 4.4.7 of Priestley et al. (2007)

5d.(iii) Concrete stress block

$$\varepsilon_c = \left[\frac{\theta L_{cant}}{\left(L_{cant} - \frac{L_p}{2}\right) L_p} + \phi_y\right] c$$

- Strain penetration length:

$$l_{sp} = 0.022 f_y d_{b,e} = 405\ mm$$

- Plastic hinge length:

$$L_p = 0.08H + lsp = 965\ mm$$

5d.(iii) Concrete stress block

$$\varepsilon_c = \left[\frac{\theta L_{cant}}{\left(L_{cant} - \frac{L_p}{2}\right) L_p} + \phi_y\right] c = 0.0216$$

- Normalized concrete compressive strain:

$$\frac{\varepsilon_c}{\varepsilon_{co}} = 9.25$$

- Assumed concrete confinement:

$$\frac{f'_{cc}}{f'_{co}} = 1.30$$

5d.(iii) Concrete stress block

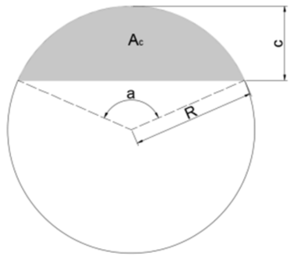
- Stress block factors for $f'_c = 50MPa$, $\frac{f'_{cc}}{f'_{co}} = 1.3$ and $\frac{\varepsilon_c}{\varepsilon_{co}} = 9.25$:

$$\alpha = 1.073$$
$$\beta = 0.9960$$

Table 16.6 Stress block coefficients for $f'_{cc} / f'_c = 1.3$, $\varepsilon_c / \varepsilon_{co} = 5.25 - 10.0$

f'c		5.25	5.5	5.75	6	6.25	6.5	6.75	7	7.25	7.5	7.75	8	8.25	8.5	8.75	9	9.25	9.5	9.75	10
30	α	1.232	1.225	1.217	1.209	1.202	1.194	1.186	1.178	1.170	1.162	1.153	1.146	1.138	1.130	1.122	1.115	1.107	1.100	1.092	**1.085**
	β	0.912	0.918	0.925	0.931	0.936	0.942	0.947	0.952	0.957	0.962	0.966	0.971	0.975	0.979	0.983	0.987	0.990	0.994	0.997	**1.001**
40	α	1.225	1.218	1.209	1.201	1.192	1.183	1.175	1.166	1.157	1.148	1.139	1.131	1.122	1.114	1.105	1.097	1.089	1.081	1.073	**1.065**
	β	0.910	0.917	0.923	0.930	0.936	0.942	0.947	0.953	0.958	0.963	0.968	0.973	0.977	0.981	0.986	0.990	0.994	0.997	1.001	**1.005**
50	α	1.220	1.211	1.203	1.193	1.184	1.175	1.165	1.156	1.146	1.137	1.127	1.118	1.109	1.100	1.091	1.082	1.073	1.065	1.056	**1.048**
	β	**0.908**	**0.916**	**0.923**	**0.929**	**0.936**	**0.942**	**0.948**	**0.953**	**0.959**	**0.964**	**0.969**	**0.974**	**0.979**	**0.984**	**0.988**	**0.992**	**0.996**	**1.000**	**1.004**	**1.008**

5d.(iii) Concrete stress block

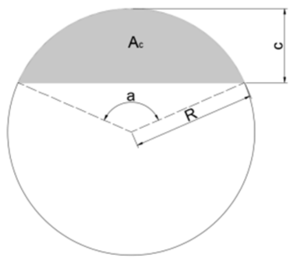


- Concrete compression area:
  $A_c = 913000\ mm^2$

- Concrete compression force:
  $C_c = 0.90 \alpha f_c' \beta A_c$
  $= 45600kN$

$$a = 2cos^{-1}(1 - \frac{c}{R})$$
$$A_c = \frac{R^2}{2}(a - sina)$$

5d.(iv) Check force balance



- Check force balance:
  $\Sigma F = N + T_{PT} + \Sigma T_s(i) - C_c$
  $= 1540kN \neq 0$ ✗

- Therefore iteration on c is required.

$$A_c = \frac{N + T_{PT} + \Sigma T_s(i)}{0.90 \alpha f_c' \beta}$$
$$= 881700\ mm^2$$

$$c = 634\ mm$$

$$a = 2cos^{-1}(1 - \frac{c}{R})$$
$$A_c = \frac{R^2}{2}(a - sina)$$

5d.(v) Final Iteration

- $c = 637\ mm$
- $\Delta_{PT} = n\theta\left(\frac{D}{2} - c\right) = 11.28\ mm$
- $T_{PT,i} = 16560\ kN$
- $l_{ub,PT} = 6000\ mm$
- $\Delta\varepsilon_{PT} = \frac{\Delta_{PT}}{l_{ub,PT}} = 1881\mu\varepsilon$
- $\Delta T_{PT} = \Delta\varepsilon_{PT} E_{PT} A_{PT} = 5230\ kN$
- $T_{PT} = T_{PT,i} + \Delta T_{PT} = 22500\ kN$ $\left(92\%\ T_{PT,y}\right) - Assumed\ to\ be\ OK$

Summary of mild steel bar forces

$Layer, i$	$d(i)$	$n_b(i)$	$A_s(i)$	$\Delta_s(i)$	$\varepsilon_s(i)$	$f_s(i)$	$T_s(i)$
	mm		mm²	mm	με	MPa	kN
1	70	1	881	-15.5	-19819	-577	-509
2	76	2	1763	-15.3	-19609	-577	-1017
3	95	2	1763	-14.8	-18944	-576	-1015
4	125	2	1763	-14.0	-17895	-574	-1012
5	167	2	1763	-12.8	-16425	-572	-1008
6	220	2	1763	-11.4	-14571	-569	-1003
7	284	2	1763	-9.62	-12332	-565	-997
8	357	2	1763	-7.63	-9778	-561	-989
9	439	2	1763	-5.39	-6910	-557	-981
10	529	2	1763	-2.93	-3761	-552	-972
11	625	2	1763	-0.31	-403	-81	-142
12	726	2	1763	2.44	3131	551	971
13	832	2	1763	5.33	6839	557	981
14	940	2	1763	8.28	10617	563	992
15	1050	2	1763	11.3	14465	569	1003
16	1160	2	1763	14.3	18314	575	1013
17	1268	2	1763	17.2	22092	581	1024
18	1374	2	1763	20.1	25800	587	1035
19	1475	2	1763	22.9	29333	593	1045
20	1571	2	1763	25.5	32692	598	1054
21	1661	2	1763	28.0	35840	603	1063
22	1743	2	1763	30.2	38709	608	1071
23	1816	2	1763	32.2	41263	612	1078
24	1880	2	1763	33.9	43502	615	1084
25	1933	2	1763	35.4	45356	618	1090
26	1975	2	1763	36.5	46825	621	1094
27	2005	2	1763	37.3	47875	622	1097
28	2024	2	1763	37.9	48540	623	1099
29	2030	1	881	38.0	48749	624	550
						$\Sigma T_s(i)$	8700

5d.(v) Final Iteration

- $\varepsilon_c = \left[\frac{\theta L_{cant}}{\left(L_{cant} - \frac{L_p}{2}\right) L_p} + \phi_y\right] c = 0.0212$
- $\frac{\varepsilon_c}{\varepsilon_{co}} = 9.06$
- $\alpha = 1.080$
- $\beta = 0.9930$
- $A_c = 886400\ mm^2$
- $C_c = 0.90\alpha f_c' \beta A_c = 44500 kN$
- $\Sigma F = N + T_{PT} + \Sigma T_s(i) - C_c = 0\ kN$ ✓

5d.(vi) Moment Capacity

- Depth to concrete compression force:

$$\frac{a}{2} = \frac{\beta c}{2} = 316\ mm$$

- Gravity load moment contribution:

$$M_N = N\left(\frac{D}{2} - \frac{a}{2}\right) = 9760\ kNm$$

- Post-tensioning moment contribution:

$$M_{PT} = T_{PT}\left(\frac{D}{2} - \frac{a}{2}\right) = 16500\ kNm$$

5d.(vi) Moment Capacity

- Mild steel moment contribution per layer:

$$M_{ms}(i) = \Sigma T_s(i)\left(d(i) - \frac{a}{2}\right) = 22900\ kNm$$

- Moment capacity:

$$M_n = M_N + M_{PT} + \Sigma M_{ms}(i) = 49200\ kNm\ ✓$$

- Recentering ratio:

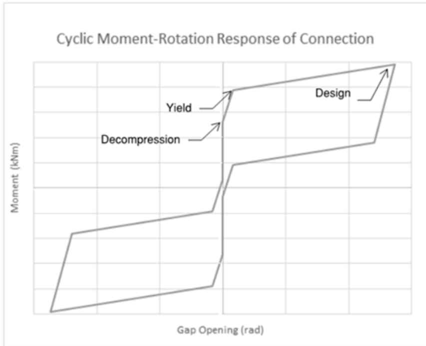
$$\lambda = \frac{M_N + M_{PT}}{M_{MS}} = \frac{9830 + 16500}{22900} = 1.15\ ✓$$

Summary of mild steel moment contributions

Layer, i	$d(i)$	$T_s(i)$	$M_{ms}(i)$
	mm	kN	kNm
1	70	-509	125
2	76	-1017	244
3	95	-1015	224
4	125	-1012	193
5	167	-1008	150
6	220	-1003	96
7	284	-997	32
8	357	-989	-41
9	439	-981	-121
10	529	-972	-207
11	625	-142	-44
12	726	971	398
13	832	981	506
14	940	992	619
15	1050	1003	736
16	1160	1013	855
17	1268	1024	975
18	1374	1035	1095
19	1475	1045	1211
20	1571	1054	1323
21	1661	1063	1430
22	1743	1071	1528
23	1816	1078	1617
24	1880	1084	1696
25	1933	1090	1762
26	1975	1094	1815
27	2005	1097	1853
28	2024	1099	1877
29	2030	550	942
		$\Sigma M_{ms}(i)$	22900

5e. Determine cyclic response

i. Decompression Moment
ii. First Yield
iii. Ultimate Limit State
iv. Maximum Considered Event



5e. Decompression Moment

- The decompression moment is that which first induces tensile stress in the connection – i.e. initiates gap opening.
- The relationship between the stresses in the section are related by:

$$\frac{M_{dec}}{Z} - \frac{T_{pt,i}.e}{Z} - \frac{T_{pt,i} + N}{A} = 0$$

Where e is any eccentricity between the post-tensioning location and the centroid of the section (zero in this case).

5e. Decompression Moment

$$M_{dec} = \left(\frac{T_{pt,i}.e}{Z} - \frac{T_{pt,i} + N}{A}\right).Z = 11450\ kNm$$

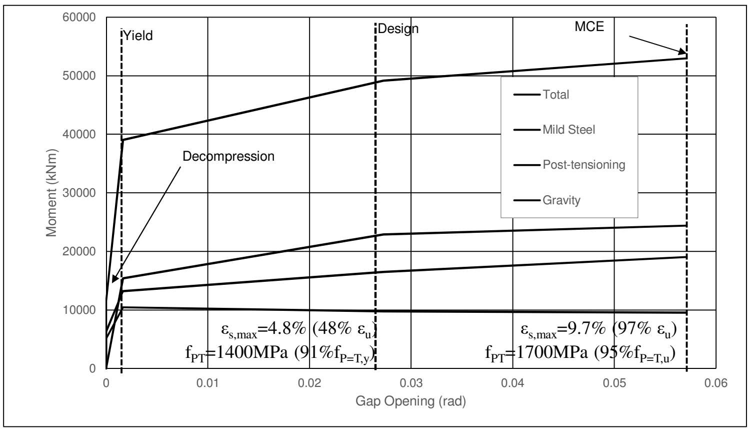
- Base curvature: $\phi_{col} = \frac{M_n}{EI} = 0.389\ \times 10^{-3}\ m^{-1}$
- Rotation at top of column: $\theta_{col} = \frac{\phi_{col} H_c}{2} = 0.895\ \times 10^{-3}\ rad$
- Elastic deflection under decompression moment:

$$\Delta_e = \frac{2\theta_{col} H_c}{3} + \theta_{col}(H - H_{c)} = 4.89\ mm$$

5e. First Yield & MCE

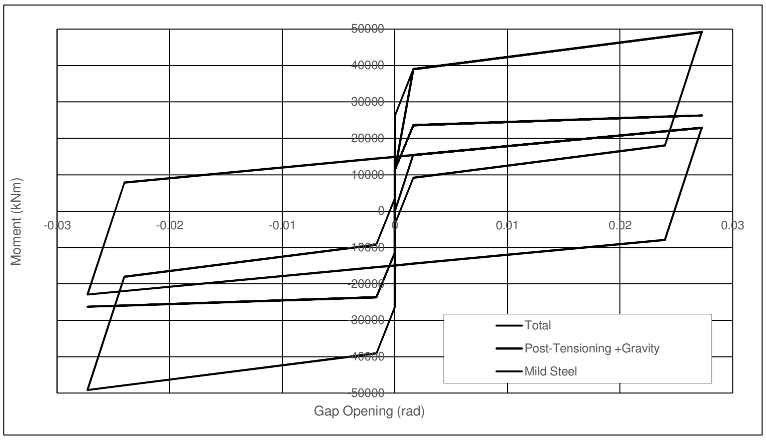
- We repeat the connection analysis for the first yield and MCE points.
- First Yield
  - Determine gap opening at which yield of the first layer of mild steel occurs
  - Calculate moment capacity at this gap opening
  - Determine elastic deflection of the column at the yield point
- Maximum Considered Earthquake (MCE)
  - Displacement under MCE case is assumed to be 2 times the ULS in this example
  - Reinforcing bar checked to ensure that strain < 10%
  - Post-tensioning checked to ensure that $T_{PT} < T_{PTU}$

5e.M-θ Backbone



Backbone Moment-Rotation Response of Connection

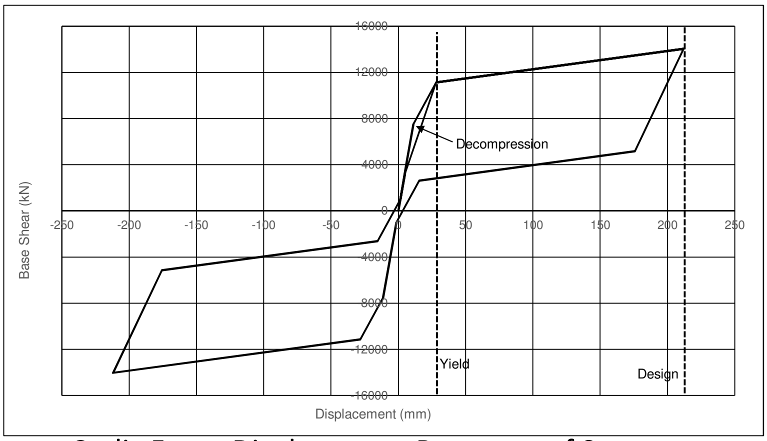
5e.Plot: M-θ Cyclic



Cyclic Moment-Rotation Response of Connection

5a. Moment and gap 5b. PT Quick 5c. MS Quick 5d. Detailed design **5e. Cyclic** 5f. Detailing

5e. Plot: F-Δ Cyclic



Cyclic Force-Displacement Response of Structure

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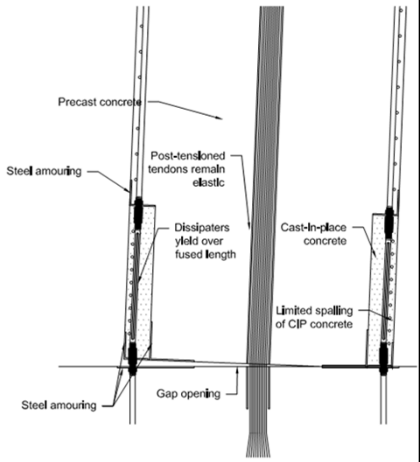
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

5a. Moment and gap 5b. PT Quick 5c. MS Quick 5d. Detailed design 5e. Cyclic **5f. Detailing**

5f. Detailing

- Energy dissipaters
- Coupled connection
- Armouring and stirrups
- Joint shear
- Durability

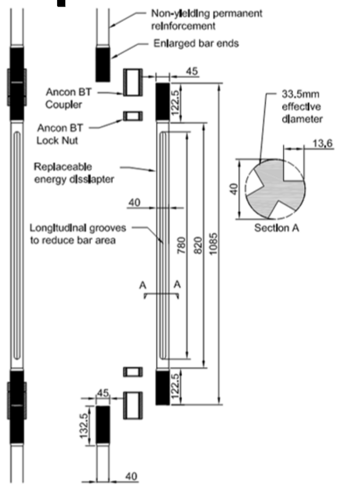


NZCS New Zealand Concrete Society

Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

5f.(i) Energy dissipaters

- 'Grooved' dissipaters used
- Area reduced by milling longitudinal grooves
- Radius of gyration is 11% higher than dissipaters with circular cross section of equal area
- Sleeve can be added to further resist buckling. Constant outside diameter along dissipater.

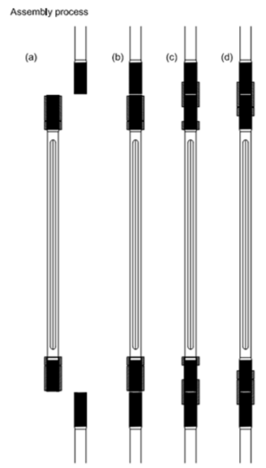


5a. Moment and gap 5b. PT Quick 5c. MS Quick 5d. Detailed design 5e. Cyclic **5f. Detailing**

5f.(ii) Coupled connection

- Ancon BT couplers
- Bar ends are enlarged through cold forging from a diameter of 40mm to 45mm
- Full strength connection of full diameter bar.
- Coupler utilization ratio

$$\frac{1.35A_b f_y}{1.35A_{b,full} f_y} = \frac{595kN}{848kN} = 70\%$$



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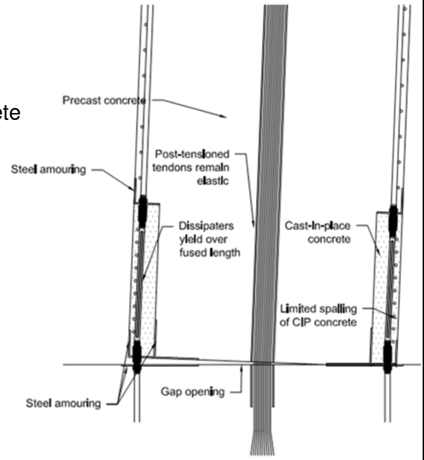
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

5f.(iii) Armouring and stirrups

- Armouring
  - Protects precast components
  - Prevents premature crushing of CIP concrete
  - Helps to position bar ends for couplers
- Stirrups
  - Confine upper part of CIP region.
- Manders confined concrete model

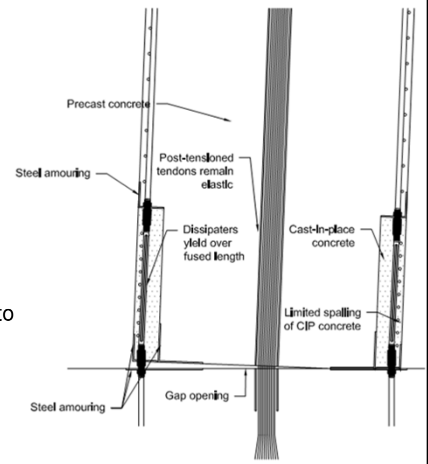
$f'_{cc}=1.3f'_{co}$



5a. Moment and gap 5b. PT Quick 5c. MS Quick 5d. Detailed design 5e. Cyclic **5f. Detailing**

5f.(iii) Joint shear

- Shear friction used in this case
- NZS3101 B6.6.4
  - Allows reliance on dowel action and shear friction capacity provided by unfactored gravity load.
- Friction coefficient: $\mu = 0.7$
  - NZS3101 7.7.4.3(d) – Concrete anchored to as-rolled steel by reinforcing bars



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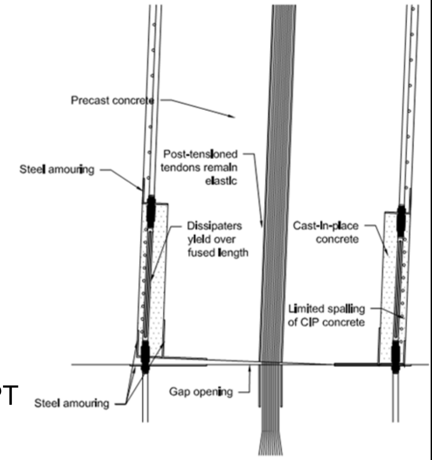
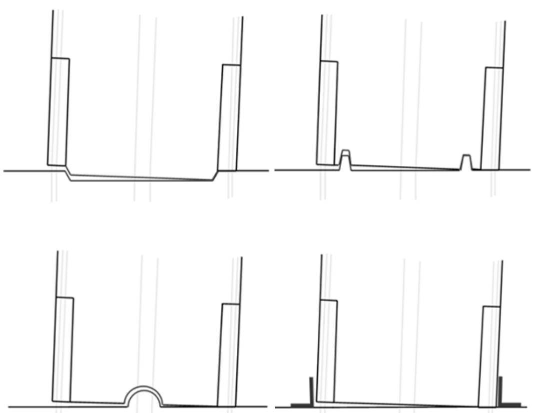
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

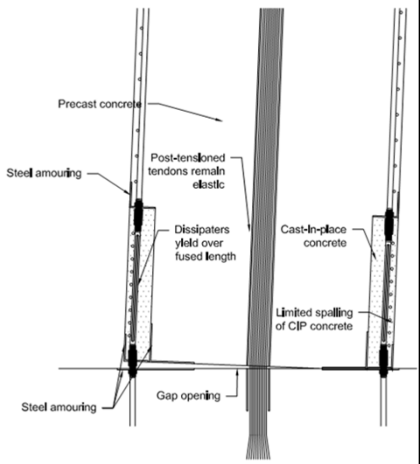
5f.(iii) Joint shear

- Unfactored gravity load:
  $$N = 13300\ kN$$
- Shear friction capacity:
  $$\Phi V_{SF} = \Phi N\mu = 0.75 * 13300 * 0.7 = 7000kN$$
- Lateral design force per pier
  $$F_{pier} = 7050\ kN$$
- Not accounting for axial load due to PT or dowel action.



5f.(iii) Joint shear





Session 5

Case Study One: Wigram-Magdala Bridge Link

Peter Routledge

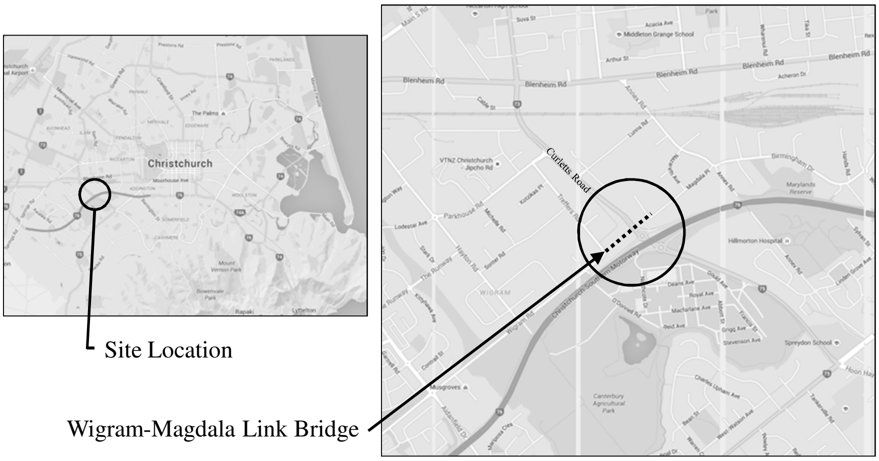
Case Study 1: Wigram-Magdala Link Bridge

Peter Routledge, Senior Bridge Engineer,
Opus International Consultants Ltd.

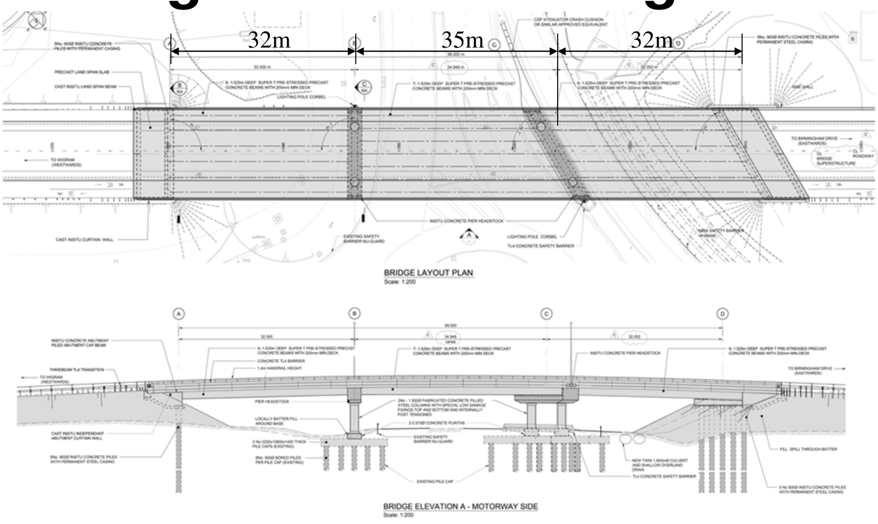
Introduction

- Description of Wigram-Magdala Link Bridge
- University of Canterbury low-damage testing which inspired the project
- Detailing issues particularly associated with the decision to 'bury' the external dissipaters
- Design Method
- Construction photos
- Lessons Learnt

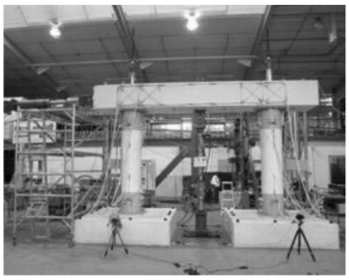
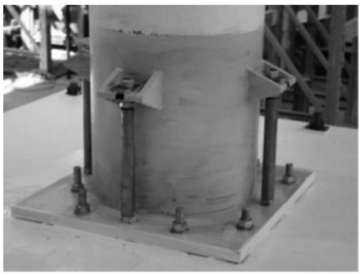
Wigram-Magdala Link Bridge



Bridge General Arrangement

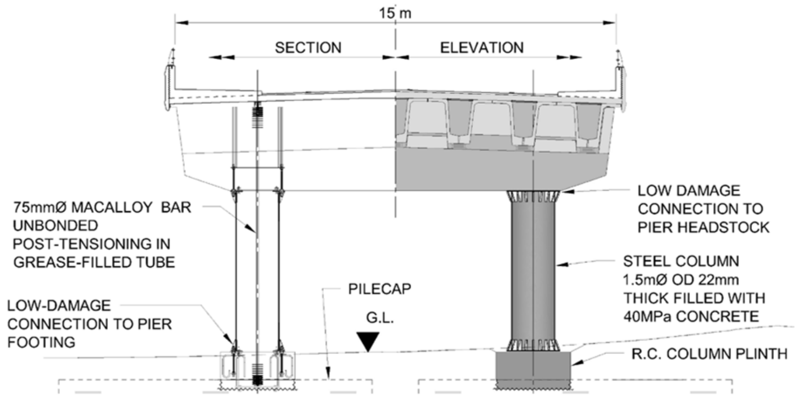


Low-damage column connection tested at University of Canterbury

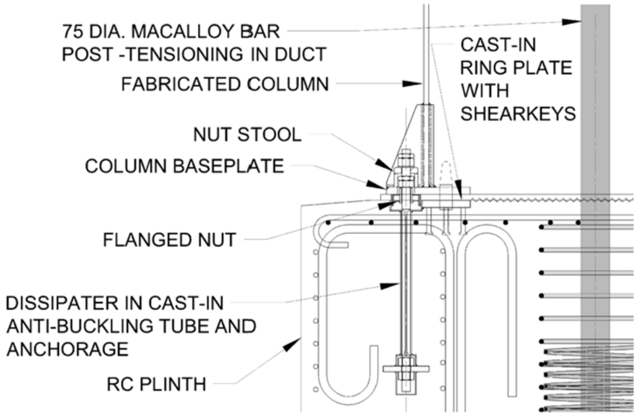




Testing was part of the Government funded project “Advanced Bridge Construction and Design” at University of Canterbury

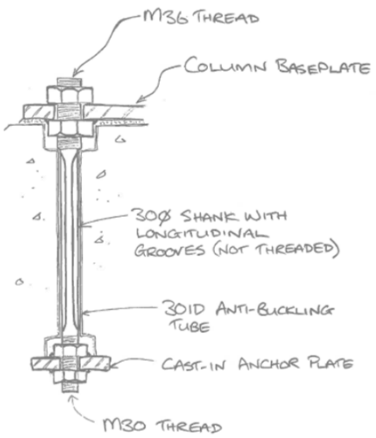
Section/Elevation on Pier



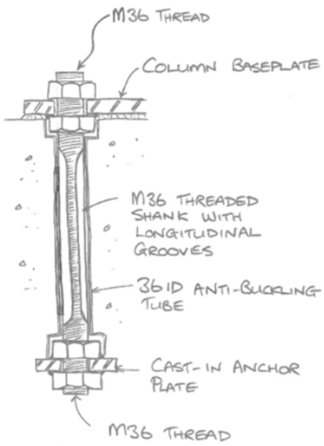
Low-damage connection between column and pier footing



Buried dissipater concepts

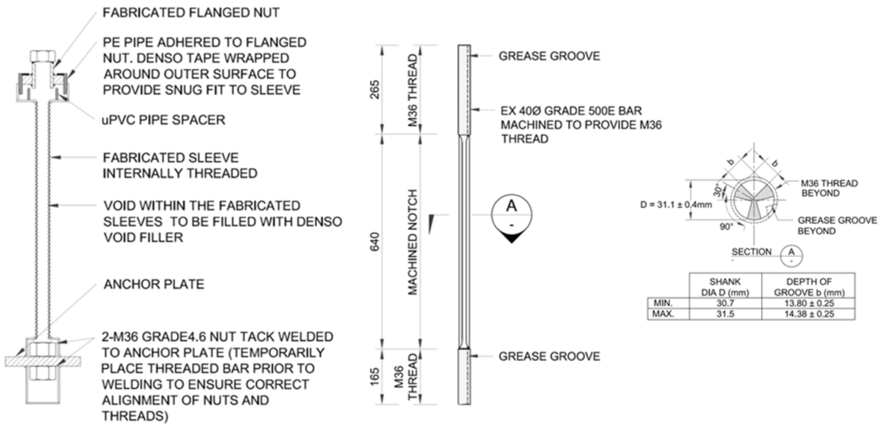


a) M36 upper anchorage with M30 lower anchorage

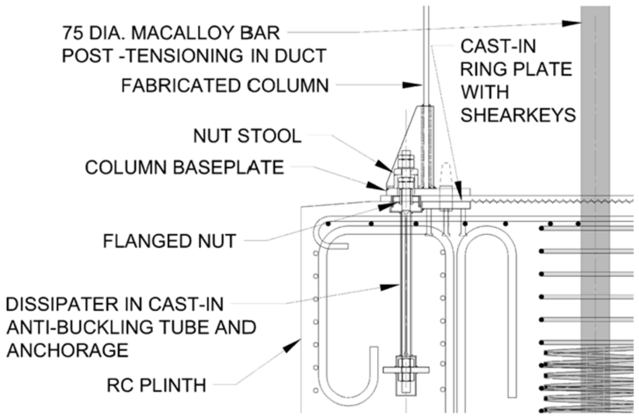


b) M36 fully threaded shank

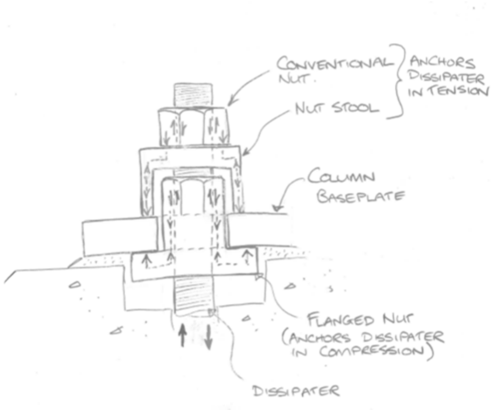
Buried dissipater details



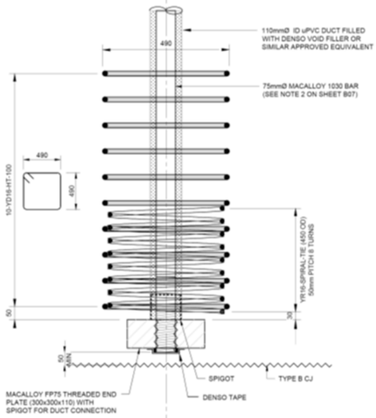
Low-damage connection between column and pier footing



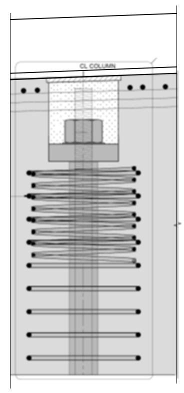
Flanged nut and load paths



Replaceable Post-tensioning



Lower PT anchorage



Upper PT anchorage

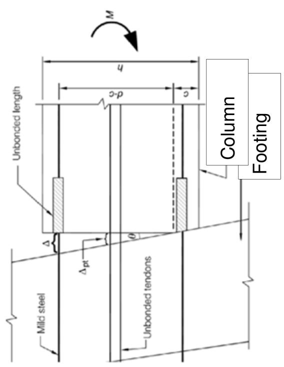
Design Method

- Designed generally in accordance with the requirements of NZS 3101 Appendix B
- Direct Displacement-Based Design procedure was used incorporating the recommendations of The Bridge Manual v3 (Draft Section 5) and the following clarifications:
  - The diameter of the hybrid joint = diameter of the column endplate
  - Dissipaters designed to yield but remain within the strain limits at ULS and MCE.
  - Post-tensioning was designed to remain elastic but permitted to yield in an event larger than ULS.
  - Self-centring was assured by providing a moment contribution ratio $\lambda \geq 1.5$ as defined by Eq B-1 (NZS 3101)
  - The refined damping equation of Eq 6.11 (PRESSS Handbook) was used as follows:

$$\xi_{eq} = 0.05 + \frac{R_\xi}{(\lambda + 1)}\left(\frac{\mu - 1}{\mu\pi}\right)$$

Where $R_\xi = 0.577$ (Ramberg-Osgood) since this is considered to represent the damping behaviour of externally mounted steel yielding dampers where the anchorage of the damper to the mounting is stiff.

Hybrid Joint



Rocking mechanism of hybrid joint

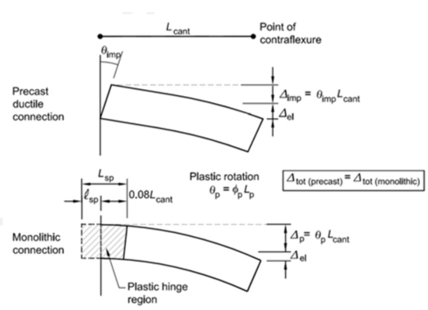


Figure CB.7 – Monolithic beam analogy for member compatibility condition

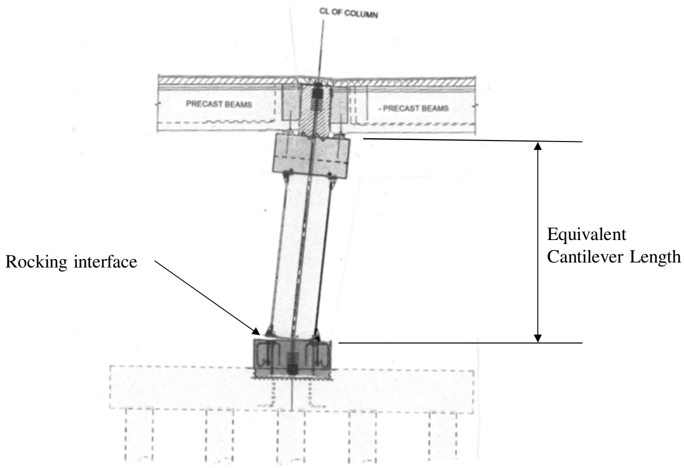
$$\varepsilon_s = \frac{\left(\Delta_s + 2/3\ell_{sp}\varepsilon_y\right)}{\left(\ell_{ub} + 2\ell_{sp}\right)}$$

$$\varepsilon_c = \left[\frac{(\theta L_{cant})}{\left(L_{cant} - \frac{L_p}{2}\right)L_p} + \phi_y\right]c \;..$$

$$\varepsilon_{pt}(\theta) = \frac{n\Delta_{pt}}{\ell_{ub}} \;..$$

Ref. NZS 3101 Appendix B

Longitudinal Behaviour



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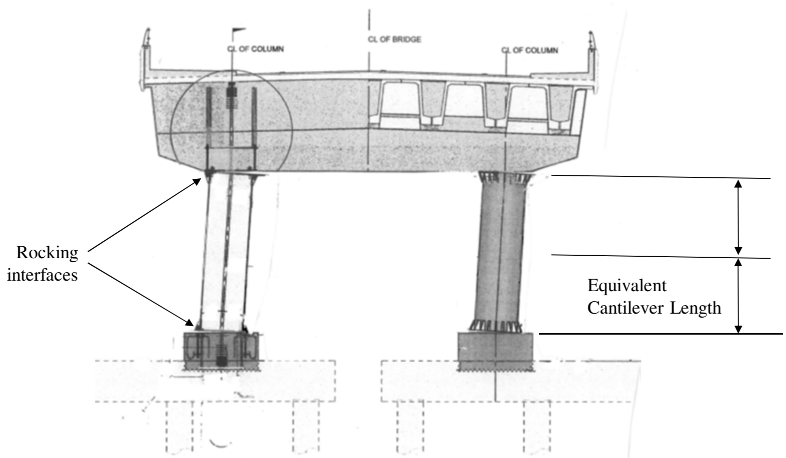
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

Transverse Behaviour



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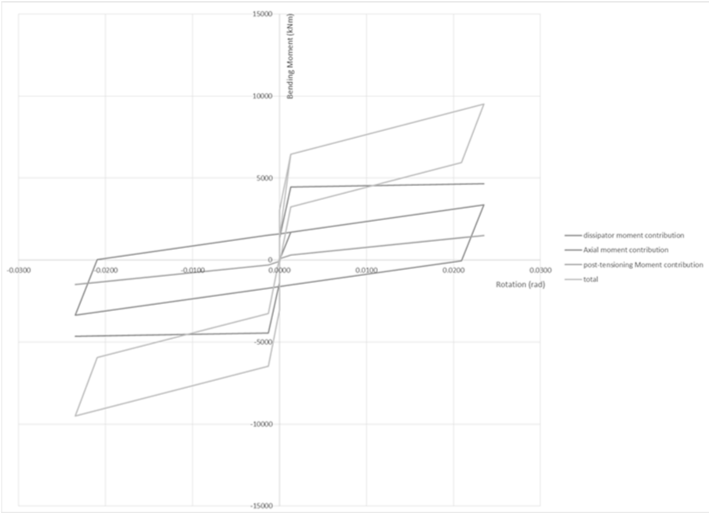
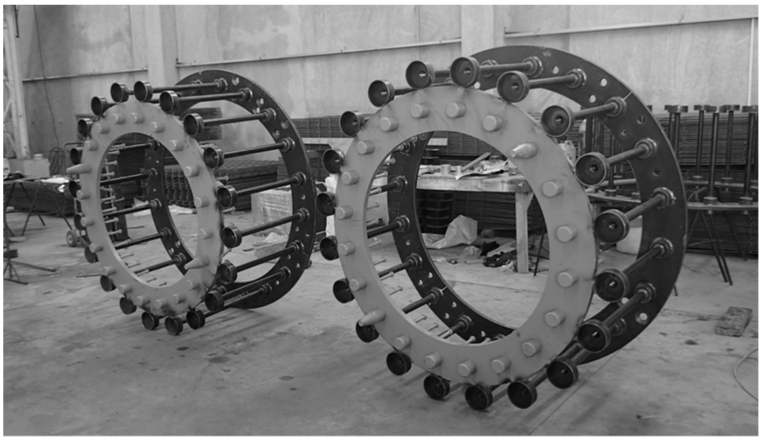
Moment Rotation Curve



Construction Photos



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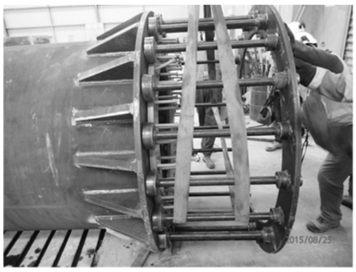
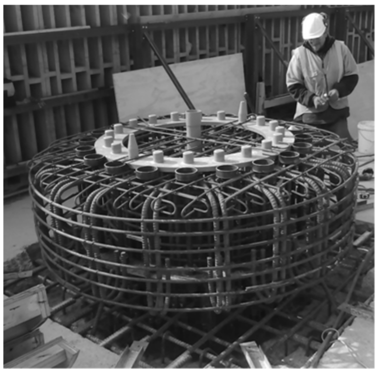
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers


2015/08/25



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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

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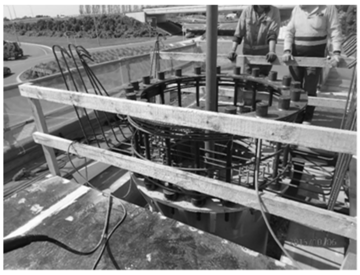

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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

Lessons Learnt

- Scaling-up details for use on a bridge – dissipater size limitations
- Machining difficulties, availability of suitable size and grade of steel for dissipaters - costly
- Burying the dissipaters created a ‘blind connection’ – detailing required a lot of thought, complexity and tight tolerances
- Economic justification may be difficult
- Column prefabrication provides programme benefits
- Further research to refine the area ratio of grooved portion to threaded portion
- The details could be compatible with connecting to prefabricated footings and crossheads – tolerances!
- Alignment of multiple threaded components
- Machining from solid billet rather than welding
- Steel column casing – structural or non-structural
- Increased efficiency of dissipaters due to increased effective depth

Conclusions

- Presented an example of some low-damage details used on a real bridge
- Connections may be very visible on a bridge -aesthetics
- The decision to bury the connection has improved the aesthetics of the low damage detail but introduced new detailing issues:
  - Ease of replacement
  - Durability and inspection
  - Buckling restraint to dissipaters
  - Dissipater profile and fabrication issues
- Whilst the details have been constructed successfully these details should be considered a step towards a solution rather than a recommended solution. Further refinements are possible.

Acknowledgements

- Christchurch City Council
- University of Canterbury
- Opus International Consultants Ltd

Session 6

Case Study Two: Mackays to Peka Peka

Jamil Khan

MacKays to Peka Peka Expressway (M2PP) Project

A Case Study

By Jamil Khan

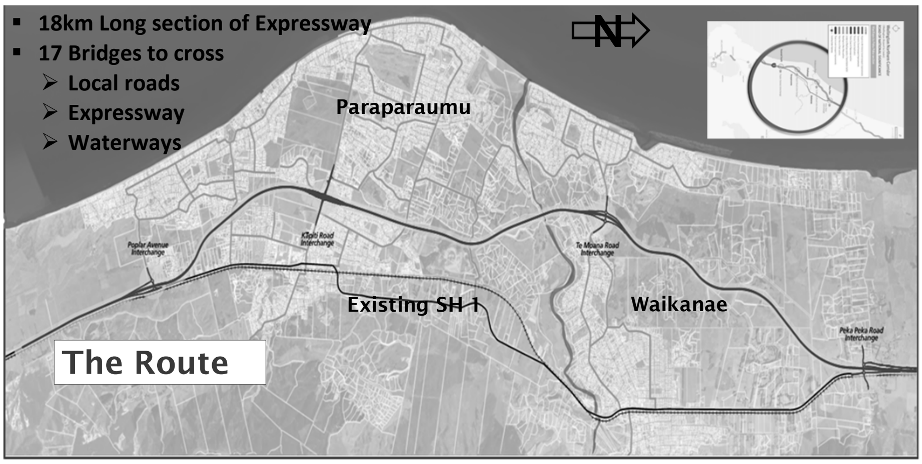
Beca

NZCS | New Zealand Concrete Society

Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

M2PP Expressway Project

Project Background



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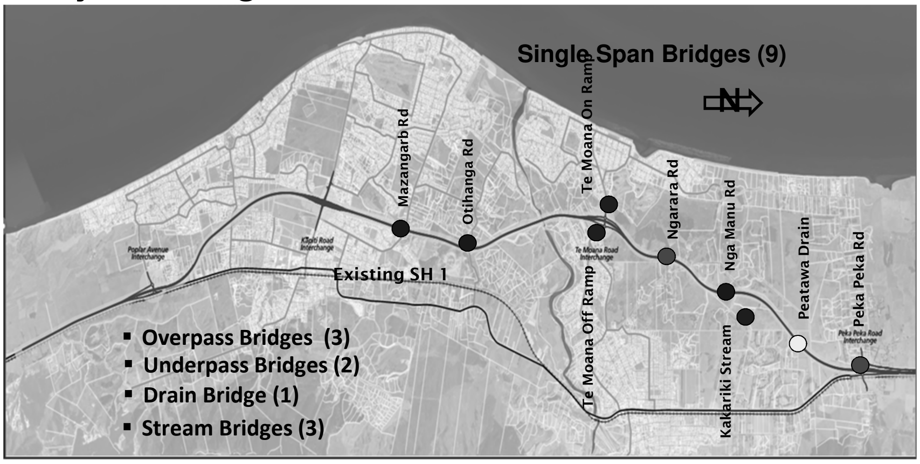
Project Background



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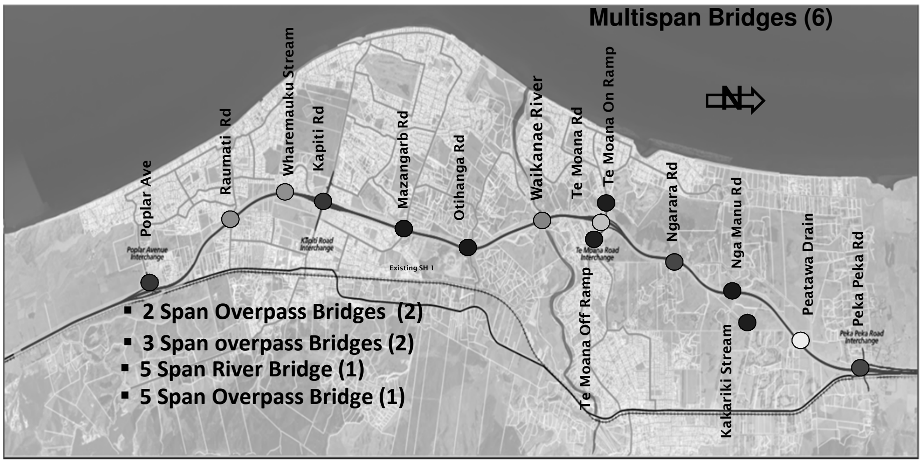
Project Background



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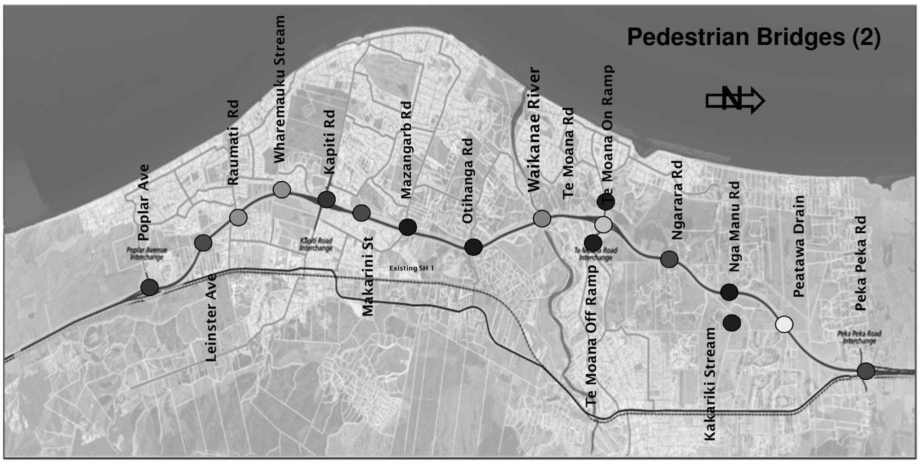
Project Background



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M2PP Expressway Project

Challenging Environment

Ground Conditions

- The Expressway passes through
  - Sand dunes and low lying inter-dunal deposits
- Peat deposits are present along alignment
  - Peat is very soft soil with very high organic content & highly compressible. These deposits are typically 0.5m to 6m thick
- Liquefiable loose sand and silt layers below GWT
- Founding layer of dense gravel sand for piles is about 20 ~30m below GL

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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

M2PP Expressway Project

Challenging Environment (Cont)

- Critical faults around the site:

- Ohariu Fault
  - Estimated MCE of M7.2
  - Return period – 2000yrs
  - 1km from Peka Peka Bridge
  - 3km from Poplar Avenue Bridge
- Wairarapa Fault
  - Estimated MCE of M8.2
  - Return period - 1200 yrs
  - 27km from Poplar Ave Bridge
  - 30km from Waikanae River Bridge

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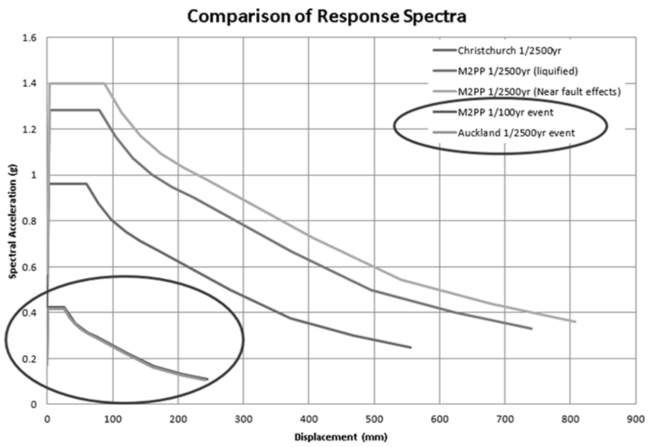
Challenging Environment (Cont)

Seismicity of the Area (cont.)



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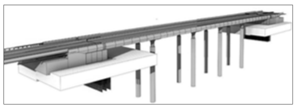
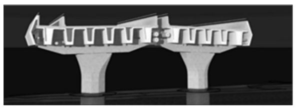
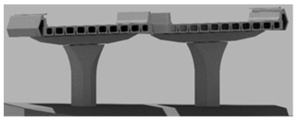
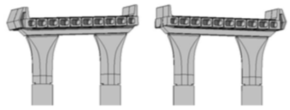
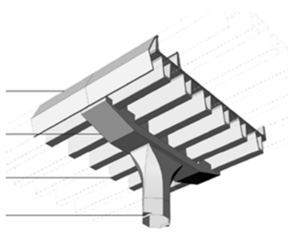
Multi-span Bridge Form

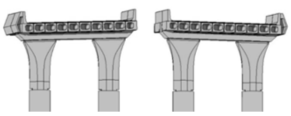





- Bridge Beams (Super T / SHC)
  - Installed as simply supported
  - Connected together via
    - Transverse post-tensioning (SHC beams)
    - Concrete topping slab (super T beams)
- Bridge Deck
  - Longitudinal direction - Bridge deck will slide over the abutments
  - Transverse direction - Deck will be restrained by shear keys
- Bridge Piers
  - Single or twin column, with precast crosshead beams
  - Supported by oversized large diameter bored piles
  - Resist total longitudinal inertia deck loads and
  - Share transverse seismic deck effects with abutments
- Bridge Abutments
  - The abutments are cast-in-situ concrete
  - Supported on steel H-piles
  - Resist only transvers seismic inertia loads of bridge deck.

M2PP Expressway Project

Accelerated Bridge Construction

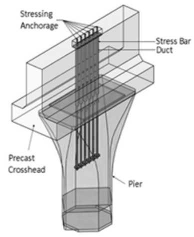


- In the design and construction of M2PP multi-span bridges, a number of “Accelerated Bridge Construction” techniques adopted by following the principals of ABC
  - Use standard elements repeatedly
    - 2 column type, 1.5m and 2.1m
    - 2 Bored Pile size 2.1m and 3m
    - 2 type of piers
    - 3 types of crosshead beams
    - 2 types of beams, Super T and SHC
  - Eliminate an element or operation if possible to reduce the number of elements / operations
    - Pier supported on large dia mono pile, no pilecap
    - Column cage plunged on pile top, no 2nd stage pour

M2PP Expressway Project

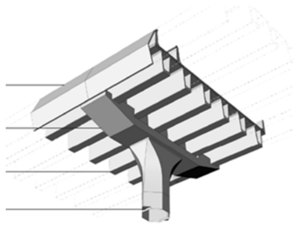
Accelerated Bridge Construction



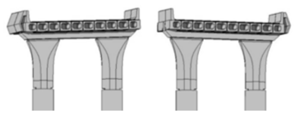
- Decouple or remove the dependency of elements on each other
  - Prefabrication of column and pile cage
  - Post-tension connection of column & crosshead
  - A short land span to separate MSE Walls and Abutment, so MSE wall can be constructed independently.
- Off-site pre-casting and prefabrication of reinforcement cage to improve the quality
  - Prefabrication of column & pile cage, abutment pile caps cage
  - Pre-cast bridge beams, crosshead beams, facia panels, facing panels, land span slabs etc
- Minimise confined space working at site
- Minimise working at height

M2PP Expressway Project

Multi-span Bridge Piers



- In an urban environment, piers are designed to:
  - Be aesthetically appealing
  - Occupy less space
  - Provide more light & room underneath bridge
- Types of pier adopted are:
  - Hammerhead pier (for 3 and more spans bridges)
  - Portal frame pier (for 2 span bridges)
- The components of piers are:
  - Monopile continuous with pier column
  - Pier column
  - Crosshead beam
  - Pier column & cross-head beam connection
  - Pier & deck linkage connection



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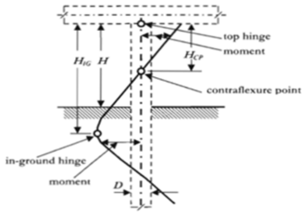
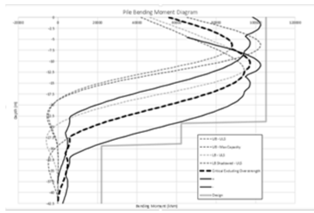
Bridge Piers Design

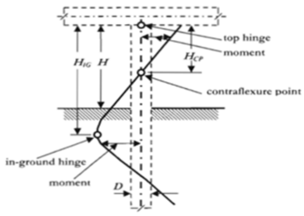


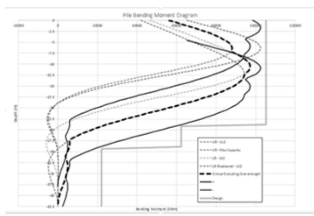


- Design economy was achieved
  - Allowing the hinges in piles at depth
  - Both the pile yield, and column yield produce a plastic mechanism that has the strength and displacement capacity to achieve the seismic demands.
  - The different mechanisms form depending on the soils characteristics (upper bound vs lower bound etc.)
  - Columns are designed for;
    - Flexural demands and not for over-strength flexure capacity of piles
    - Over-strength shear due to Plastic Hinges in piles or columns

M2PP Expressway Project

Bridge Piers Design

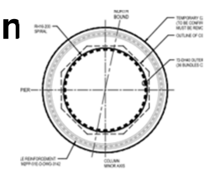




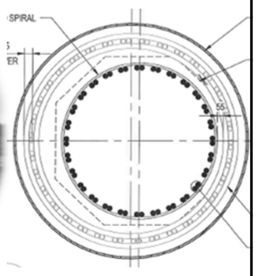
- Potential Plastic Hinges (PH)
  - Ranges of soils used for all possible PH locations
  - Potential plastic hinges' that they occur either:
    1. At the base of the column when upper bound soil conditions are considered.
    2. At a range of depths (5m-12m) when lower bound conditions considered depending on shadowing effect etc
  - Plastic hinge region is designed for Over-strength shear.

M2PP Expressway Project

Bridge Piers Column Design



- The piers column are:
  - Octagonal shape at base
  - Followed by an architectural shape
    - Elongated hexagonal shape
    - Gradually change to rectangular at top.
- The piers column cage designed as:
  - Main structural steel cage
  - Secondary steel cage
- Main structural steel cage
  - Core cage detailed as circular shape
  - Provides flexibility and tolerance
- Secondary steel cage
  - Follows the architectural shape
  - Tie to main structural steel cage



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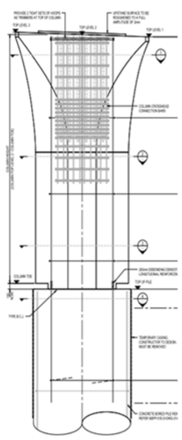
Bridge Piers Foundation Design

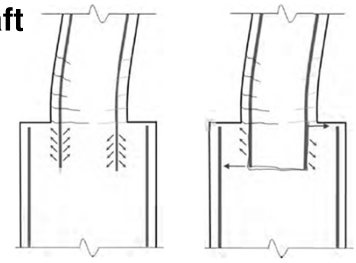


- Monopile continuous with pier
  - provides efficient support to piers
  - gives significant efficiencies in design and construction.
- Pier supported on large dia mono pile:
  - No pile cap
- Large diameter bored piles used
  - 3m dia piles for Hammered head pier
  - 2.1m dia piles for Portal frame pier
- We believe this is the1st use of 3mϕ bored pile in New Zealand.

M2PP Expressway Project

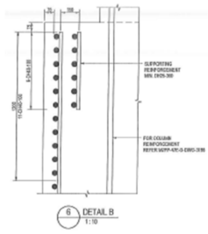
Pier Supported by Oversize Pile Shaft

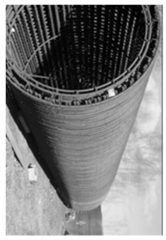
- Two types of possible failure
  - The bars pull-out of the pile.
  - Pier pull-out of the pile.
- The horizontal forces due to prying action
  - Surrounding concrete
  - Transverse reinforcement.
- No guidance in NZ Standards
- AASHTO Bridge Design Code Adopted
  - Precast Bent System for High Seismic Regionsndix A Design Provision 2013



(a) Bar pull-out mode

(b) Column pull-out mode due to plying action





M2PP Expressway Project

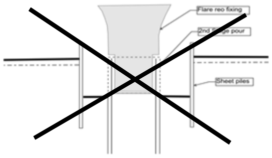
Pier & Piles Detailing (Use of DH40 Bar Hoops)

- The top of pile shaft is designed for
  - Over-strength shear demands
  - Confinement demands
  - Horizontal force demands
    - ➢ Prying action of pier supported on an oversize pile shaft
- This led team to use
  - DH40 bars in bundle for longitudinal pile reinforcement (106 bars)
  - DH40 bars as hoop reinforcement
- We believe this is the first use of HD40 bars as hoops in New Zealand.
- Pile cage weights 60 tonnes
- Constructed 32 No piles safely & efficiently

M2PP Expressway Project

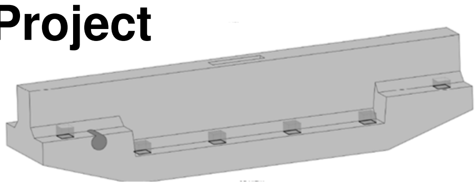
Bridge Piers and Pile Connection Design

- Column cage prefabricated
- 16T column cage plunged into the top of pile concrete
- Advantages of this innovation are:
  - No pile cap
  - No 2nd stage concreting
  - No confined space working
  - Saving the project over $6M and 120 days.
  - Produced high quality piles and pile-column interface connections

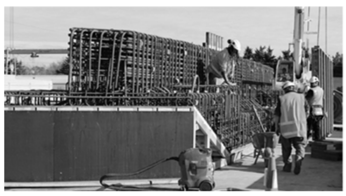


M2PP Expressway Project

Crosshead Beam



- Inverted T cantilever reinforced concrete pre-cast beams
- Designed to remain elastic under all loading cases;
  - Operational loading
  - Construction loading and
  - Imbalance of live loads.
- Design for critical load cases was undertaken to NZS3101:2006 for flexure, shear and torsion at both SLS and ULS.



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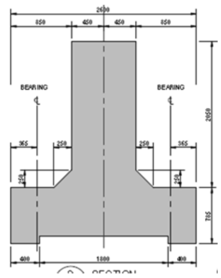
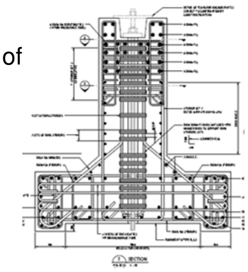
Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

M2PP Expressway Project

Crosshead Beam



- Corbel SLS design was undertaken using
  - The strut and tie methodology from BD21/01
  - Crack width calculations and requirement from NZS3101:2006 and the Bridge Manual 3rd Edition.
  - The ULS Design was undertaken to AS 5100.5-2004 Appendix D2.
- The shear keys at each end of the crosshead are designed;
  - For loading derived from transverse overstrength actions of the pier column.
  - using strut and tie theory from Appendix A of NZS3101:2006
  - Bearing stresses were assessed to section 16.3 of NZS3101:2006.



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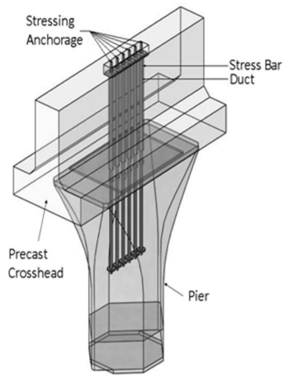
Colum - Crosshead Connection



- A Post-tension connection
- 6-56mm dia threaded stress bars was developed
  - To reduce joint congestions
  - To increase construction speed
- The steel anchorage plates with screwed threaded stress bars were cast into the column.
- The crosshead was then lowered over the column with the stress bars passing through oversize ducts, and then stressing was performed.

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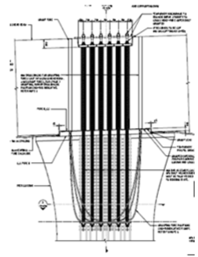
Colum - Crosshead Connection

- The PT connection was designed to remain elastic under all load cases.
- The design ULS load case results from seismic inertia generated by overstrength of the pier column.
- The SLS design actions were derived from traffic out of balance loads.
- Losses due to creep and shrinkage were accounted for in accordance to NZS3101:2006 Cl19.3.4.
- Shear transfer at the column-crosshead interface was carried through shear-friction to NZS3101:2006 Cl7.7.



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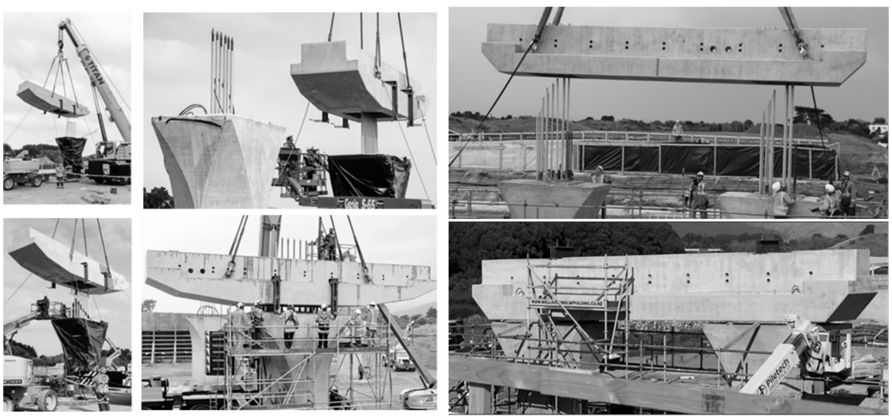
Colum - Crosshead Connection

- This allowed rapid erection of the crossheads and prove to be the most cost effective solution.



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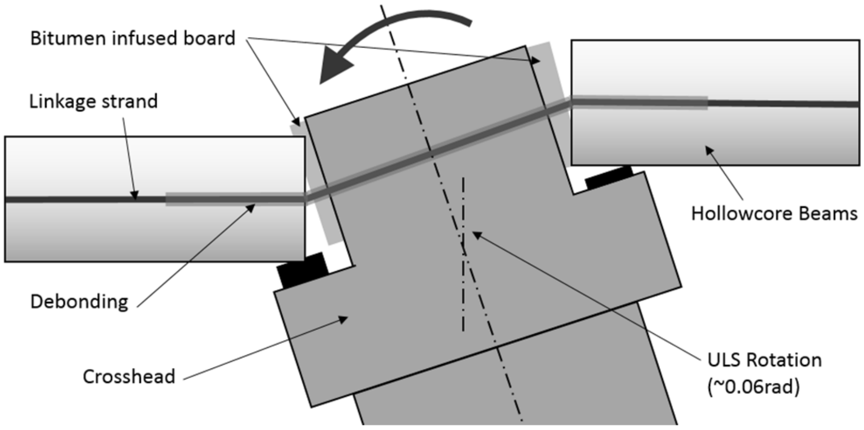
Linkage Reinforcement

- To restrain the bridge spans in the event of an earthquake.



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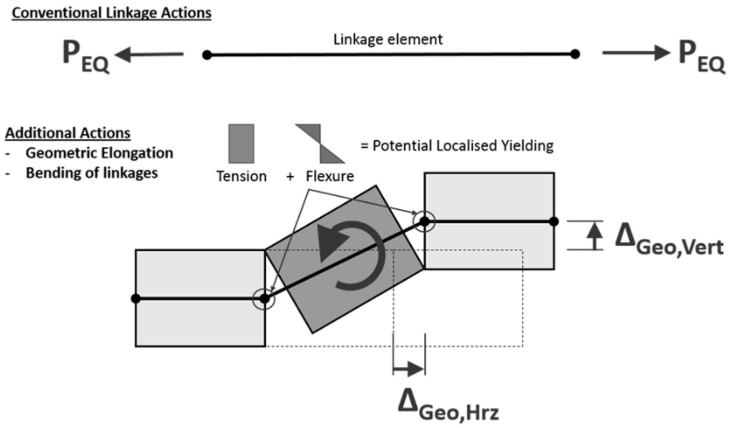
Linkage Reinforcement

- Conventional Linkage Bars
  - The linkage will yield extensively under the combined ULS action.



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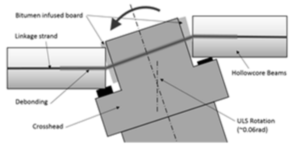
Linkage Reinforcement



- Linkage Strands Alternative
  - 15.2mm strands individually sheathed in a HDPE duct
  - Use of strand reduced flexural stresses in the linkage to a negligible value
  - Debonding increased significantly to control the strains in the linkages that develop due to geometric elongation
  - The strands are flexible enough to accommodate the rotations and axial extension without yielding/failing (both ULS and MCE).

M2PP Expressway Project

Bridge Beams

- Pre-cast pre-tension Bridge Beams
  - 650 / 900 Single Hollow Core Beam
  - 1225 /1825 Super T Beams
- Designed as partially pre-stress elements
- 1825mm Super T beam
  - Beam length 38m
  - Beam weight 88T
  - 56 numbers of 15.2mm strands
  - Jacking force 75% of Pu

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New Zealand’s First 1825 Super T Bridge Beam

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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

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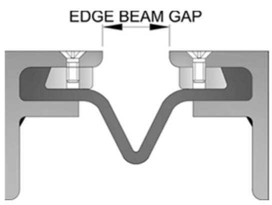
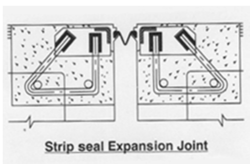
Expansion Joints

- The key considerations for an appropriate joint are;
  - Initial cost
  - Long term durability
  - Design performance
- NZTA has a strong preference for single seal joints
- In past single seal joints have provided very good long term services.
- A risk based approach adopted to gain the benefit of single seal joints.
  - List the efficiencies in construction and maintenance
  - Highlight the compromise in the design requirements.
  - List the possible risks for the Agency
  - Request for departures from Bridge Manual





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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

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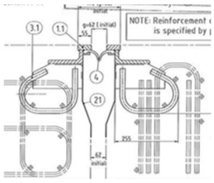
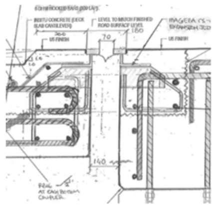
Expansion Joints, Departures from BM



- The maximum width of opening < 85mm
  - $\Delta_{TP} + \Delta_{SG} + \Delta_{GAP}$ < 85mmn
- Minor EQ return factor for expansion joints is R_s 0.39
  - $R_s = R_u / 4.23$, compare to $R_u / 4$ as suggested in BM 3rd Ed.

Bridge	Joint Type	Rs	SLS2 Event (AEP)
Waikanae River Bridge	Multi seal	0.50	1/100
Raumati Road Bridge	Multi seal	0.50	1/100
Poplar Ave Bridge	Single seal	0.50	1/100
Peka Peka Road Bridge	Single seal	0.50	1/100
Kapiti Road Bridge	Single seal	0.42	1/60
Wharemauku Stream Bridge	Single seal	0.40	1/55
Te Moana Road Bridge	Single seal	0.39	1/45



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Accelerated Bridge Construction in Seismic Areas: design detail of bridge piers

M2PP Expressway Project

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M2PP Expressway Project

Conclusions

- The high seismicity, challenging soil conditions and urban environment presented unique challenges to the Alliance team
- A number of Accelerated Bridge Construction (ABC) techniques has been adopted:
  - To improved the construction efficiencies
  - To make the safe working environment
  - To achieve the good quality product.
- Displacement based methods allowed for structures to be designed to more realistic levels of performance.
- Collaboration between designers and constructors enabled the use of a number of Accelerated Bridge Construction techniques for the multi-span bridges design.
- Innovation happens when the client, designers and constructors collaborate to put existing concepts together in a new way.
- By pushing to find ABC solutions in the design and construction of M2PP bridges, we have achieved significant construction efficiencies and savings.

M2PP Expressway Project

Acknowledgement

- The author would like to acknowledge the numerous and continuing contributions of the his designer colleagues Geoff Brown, Andrew Dickson, Ronald Wessel, Anton Kivell and the constructor colleagues Matt Zame, Ian Stockdale, Tim Pervan, Amit Chaudhary, Dave Hoffman and many others in the development and implementation of these ABC Techniques which formed the basis for the design of the M2PP Bridges.
- The author would also like to acknowledge the contributions of other Alliance partners and suppliers in the development and implementation of the Accelerated Bridge Construction approaches taken on this project.

NZ TRANSPORT AGENCY WAKA KOTAHI | Fletcher CONSTRUCTION | HIGGINS | Boffa Miskell | BRIAN PERRY CIVIL | ReoFab Limited | GOODMANS

Appendix 1

List of Reference Papers

List of Reference papers

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Appendix 2

Quasi-Static Experimental Testing of Emulative and Low-Damage Seismic Technologies for Accelerated Bridge Construction (ABC) in Seismic Areas

Mustafa Mashal
Alessandro Palermo

QUASI-STATIC EXPERIMENTAL TESTING OF EMULATIVE AND LOW-DAMAGE SEISMIC TECHNOLOGIES FOR ACCELERATED BRIDGE CONSTRUCTION (ABC) IN SEISMIC AREAS

Mustafa Mashal, University of Canterbury, (805)259-8136, mustafa.mashal@pg.canterbury.ac.nz
Dr. Alessandro Palermo, University at Canterbury, (+64)336-4298, alessandro.palermo@canterbury.ac.nz

ABSTRACT

This paper presents experimental testing and results so far for two half-scale fully precast bents. The first bent incorporates emulative cast-in-place connections to achieve plastic hinging in the columns. It is called “ABC High Damage”. For the second bent, the plastic hinging zones are armored and replaced by a combination of post-tensioning with the external replaceable dissipaters. This solution has superior seismic performance and is called “ABC Low Damage”. A comparison of both solutions is presented.

INTRODUCTION

The traditional seismic design of bridge substructure system for cast-in-place construction has been mostly relied on formation of plastic hinges at the specified locations in the columns. These locations are detailed such that the bridge will have a ductile behavior during a big earthquake. The plastic hinges are intended to absorb seismic energy by yielding of the column longitudinal bars. Therefore, plastic hinging zones in the column may suffer extensive damage such as spalling of concrete, distributed cracking, yielding and possible fracture of the longitudinal rebars, and residual displacement of the bridge following the seismic event as shown by Palermo and Pampanin(1). Using the seismic design philosophy discussed above, the bridge must remain functional and open to traffic following a design level earthquake. However, the extent of damage in the columns may be vast and therefore, extensive repair work or possible replacement of the bridge would be needed. Over the past several years, there has been increasing attention given to prefabrication of the bridge substructure systems. A notable example is research into standardized precast substructure systems by Billington et al (2). General background on ABC around the world can be found in Palermo and Mashal (3). There has also been significant interest in ABC for bridge substructure systems by the Federal Highway Administration (4) and United States Departments of Transportation such as Khaleghi (5), Ralls et al (6), Burkett et al (7), and , Utah DOT (8).

Accelerated Bridge Construction has already been implemented in regions of low seismicity in the United States (Figure 1). However, lessons learned from the past earthquakes, Hawkins et al (9) and Buckle (10), have shown vulnerability of the precast connections in high seismicity.

Figure 1. Precast Hammer Head Caps in Bell County on State Highway 36 over Lake Belton in Texas, USA

Restrepo et al (11) and Marsh et al (12) present a summary of the precast connections, suitable for use in ABC applications in seismic regions. These connections are either emulative cast-in-place or non-emulative. A range of emulative connections that can be used for ABC in seismic regions such as bar coupler, grouted duct, member socket, pocket, and integral connections have been discussed. Emulative connections aim to achieve the same seismic performance as can be expected from a cast-in-place construction. Marsh et al (12) also proposes a concept for Highways for LIFE precast bent for seismic regions (Figure 2, Left). In this type of bent, the precast column to footing connection is member socket connection, the column to cap beam connection is grouted duct.

The report by Marsh et al (12) concludes that there is still significant work underway and more needed to ensure the robustness, required seismic performance, constructability, cost effectiveness, durability, and inspectability for the proposed connections.

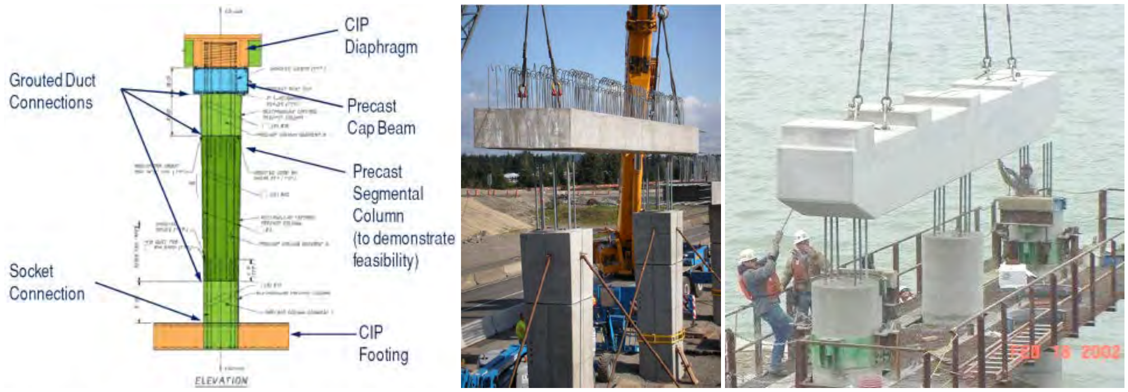


Figure 2. (Left) Highways for LIFE precast bent after Marsh et al (12), (Middle and Right) Examples of precast bent in the United States

Thus, application of ABC in high seismicity requires in depth research work and solutions, which are experimentally tested and validated. To investigate the seismic performance of ABC in high seismicity, there is an on-going major research project titled “Advanced Bridge Construction and Design” (ABCD), funded by New Zealand Natural Hazards Research Platform (2011-2015) at the University of Canterbury.

PROTOTYPE STRUCTURE

The prototype bridge is a typical 16 meters span highway bridge in New Zealand (Figure 3). The superstructure is precast I-Beam 1600 deck from the NZTA 364 (13). The substructure system is composed of piers and abutments. The piers are multi-column portal frames (bent). For simplicity, the prototype bridge is assumed to have equal spans with equal pier heights.

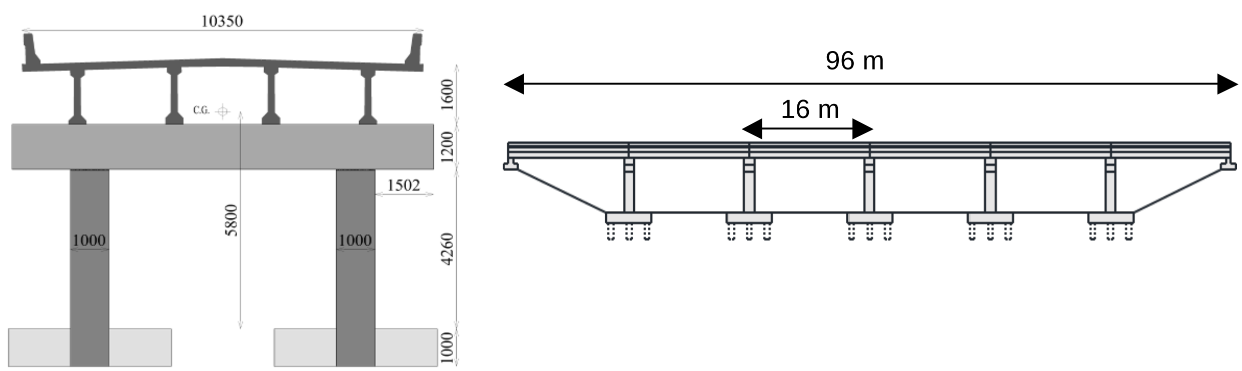
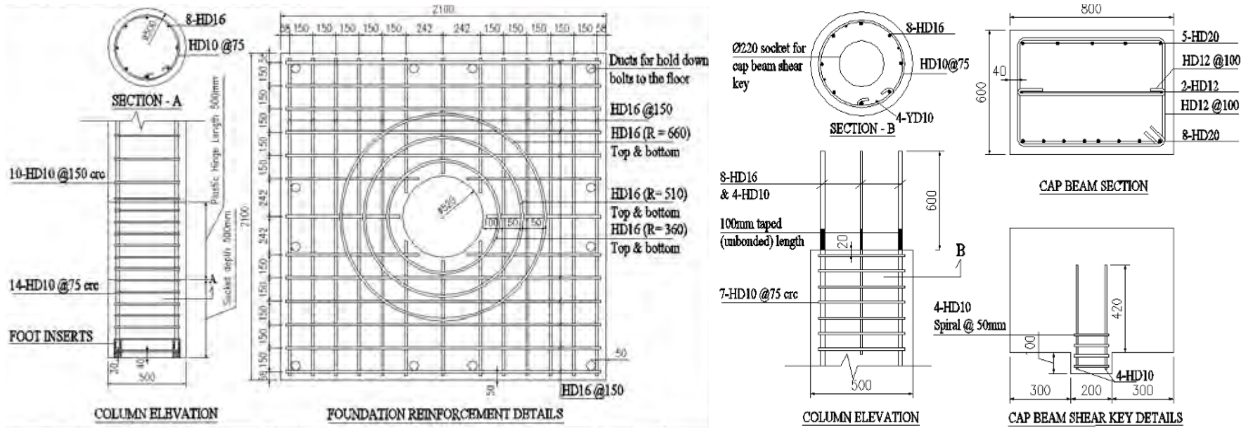


Figure 3. Prototype structure: (Left) Transverse section, (Right) Longitudinal profile

In the bent structure shown in Figure 3 (Left), the columns are circular cross section with a diameter of 1.0 meter. The footings shown above are for indicative purposes. For testing purposes, it is assumed that the footings are fully fixed. According to NZTA Bridge Manual (14) for the earthquake resistant design of the prototype bridge shown in Figure 3, the energy dissipation system relies on a ductile or partially ductile substructure. The plastic hinging is expected to happen at the design load intensity in the top and bottom of columns. The plastic hinges will form above the ground or normal water level. According to NZTA Bridge Manual (14), for the seismic design of the bent shown in Figure 3, the maximum allowable design displacement ductility during a maximum considered earthquake level has been limited to 6.0.

1ST SPECIMEN: EMULATIVE SOLUTION (ABC HIGH DAMAGE)

The specimen is a half scale bent of the prototype shown in Figure 3 (Left). It is similar to concept for Highways for LIFE precast bent by Marsh et al (12). However, the columns are not segmental in this research. The column to footing connection is member socket. The column to cap beam connection is grouted duct connection. The specimen is designed and detailed to achieve plastic hinging at the base and top of the columns, with no damage to the footings and cap beam. The columns are designed according to NZS 3101 (15). This type of solution is sometimes called emulative cast-in-place. The intended seismic performance for such a solution relies on formation of plastic hinges in the column at a design level earthquake. Therefore, moderate to extensive damage to the columns including possible residual displacement of the bridge after a big earthquake can be expected, Palermo and Pampanin (1). However, the bridge needs to remain functional after the event as discussed earlier. In this research, this type of solution which uses emulative cast-in-place connections, but has the advantage of prefabrication, is called “Accelerated Bridge Construction High Damage” or simply “ABC High Damage”. Figure 4 shows details of the precast elements for the bent. In Figure 4 all dimensions are given in mm.



(a) (b) (c) (d)

Figure 4. (a) Column bottom section and details, (b) Member socket footing reinforcement and details, (c) Column top section, starter bars, and details, (d) Cap beam section, shear key, and details

For the half-scale specimen, the gravity loads include self weight of the bent and dead load of the superstructure (390 kN). The seismic design loading for the specimen is according to NZTA Bridge Manual (14) for soil class A and B (strong rock), return period of 2500 years (bridges of high importance), zone factor of 0.3, and an assumed ductility of 3.0 at Ultimate Limit State (ULS). This yields to base shear coefficient of 0.706 (base shear of 330 kN) using an equivalent static procedure from NZTA Bridge Manual (14). The design displacement at ULS is calculated to be 60 mm or 2.1 % drift from the modal response spectrum method, as outlined in Section 5.2.6 of NZTA Bridge Manual (14).

Column to Footing Connection: Member Socket Connection

The main considerations are the socket depth, column diameter, development length of the column longitudinal bars, and the socket diameter relative to the column diameter. Foot inserts (heads) can be used to achieve the full development length of the longitudinal bars in the column as shown in Figure 5 (Left). This has the advantage of not necessarily increasing the socket depth, in order to comply with the requirements for the development length of rebars according to building codes. A 10 mm gap between the footing socket and the circular column is provided (Figure 4a and 4b). This gap will later get filled with high strength grout. In order to increase the bond between the column and the socket, the inner surface of the socket and outer surface of the column end are roughened during the construction process at the prefabrication yard (Figure 5, Right).

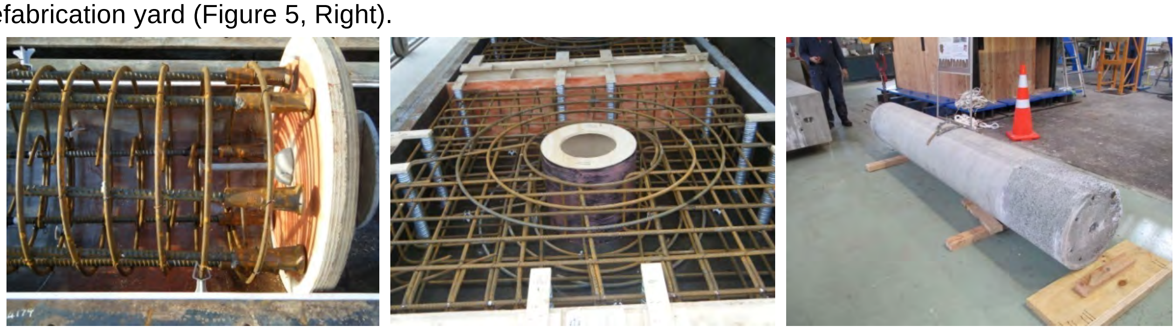


Figure 5. (Left) Foot inserts for column rebars, (Middle) Circular reinforcement around the socket, (Right) Column with roughened surface

Lateral loads induce bearing stresses in the grouted interface in the member socket connection (Figure 6, Right). A load couple forms in the socket under the lateral loading of the structure as shown in Figure 6 (Middle). Increasing the socket depth increases the distance between the coupled loads, implying less bearing force is required to overcome the moment caused by the lateral loading. Insufficient socket depth leads to bearing loads in the interface that exceed the grouts bearing capacity, causing compressive failure of the grout. The bearing stresses induced in the footing by the lateral loads are shown in Figure 6 (Right). It can be seen that accompanying the compressive stresses in the radial direction, are hoop tensile stresses that lie at a perpendicular direction to the compressive bearing stresses. This tensile stress field causes radial cracks to form which originate at the socket and propagate to the outside of the footing. This cracking can be mitigated by providing reinforcement orientated in the direction of these tensile hoop stresses. This can be achieved by providing circular hoops in the footing or hoops orientated tangentially to the hoop stresses as shown in Figure 5 (Middle).

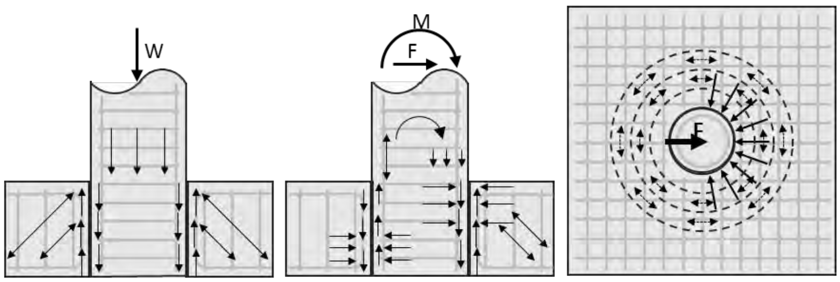


Figure 6. Internal actions in the member socket connection under: (Left) Vertical loading, (Middle) Lateral loading, (Right) Plan view showing radial compressive and tensile hoops stresses under lateral loading

Column to Cap Beam Connection: Grouted Duct Connection

Concrete circular shear keys are provided in the cap beam (Figure 7a and 7b). There is a recess for the cap beam shear key in the column (Figure 7d). The recess diameter is slightly bigger (10 mm) than the shear key. This is due to construction tolerances. The gap will be filled with gout after the assembly to emulate the conventional cast-in-place behavior of the cap beam to column connection. Note that in Figure 7c, the duct inside the shear key is left for post-tensioning in the second phase of testing for ABC Low Damage. For ABC High Damage, this duct is not utilized, since there is no post-tensioning involved.

(a) (b) (c) (d)

Figure 7. (a) Cap beam cage, (b) Cap beam with shear keys, (c) Cap beam shear key and corrugated ducts close-up view, (d) Un-bonded length of the column starter bars

The main considerations for the grouted duct connections are the shear transfer across the grouted duct connection and un-bonded length of the rebars at the column to cap beam interface.

Shear is transferred across the grouted duct connections through a combination of friction and bond in the grouted interface, and bearing of the column against the shear keys (Figure 8, Left). For design purposes, it was assumed that the shear load is transferred only through the shear keys. The shear key was designed using the methods outlined in NZS 3101 (13), treating the shear key as a corbel. The column rebars are extended into the corrugated steel ducts in the cap beam. After the ducts are grouted, the rebars are confined by the corrugated ducts and the surrounding grout (Figure 8, Middle). This means the development length needed for the starter bars is shorter. More details are given in Brenes et al (16).

The un-bonded length of the starter bars at the column to cap beam interface is intended to reduce the effect of strain penetration by distributing the longitudinal deformation of the starter bar over a longer length (Figure 8, Right). This can lead to lower levels of strain in the starter bar. Thus, it reduces the effects of low cycle fatigue and enhances the ductility. The un-bonded length can be calculated using the

NZCS PRESSS Design Handbook (17) and Priestley and Park (18). Previous testing on the grouted duct connections by Mashal et al (19) has shown that the ductility of the concrete column can be enhanced by leaving the un-bonded length at the interface between the two precast elements. By leaving the un-bonded length, the interface between the column and cap beam activates a rocking mechanism also known as gap opening. The un-bonded length can also result in having less concrete spalling in the column plastic hinging zone, as previously shown by Kawashima et al (20) and Mashal et al (19).

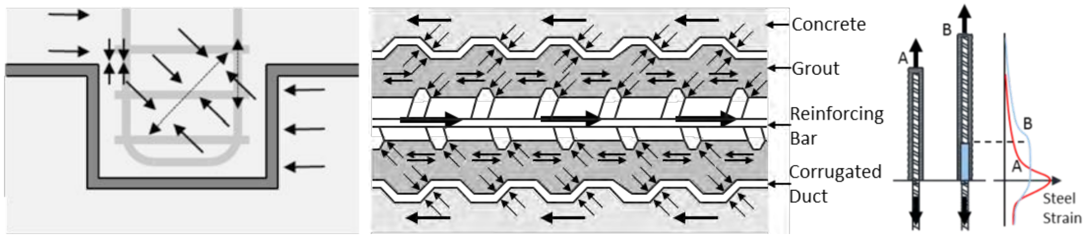
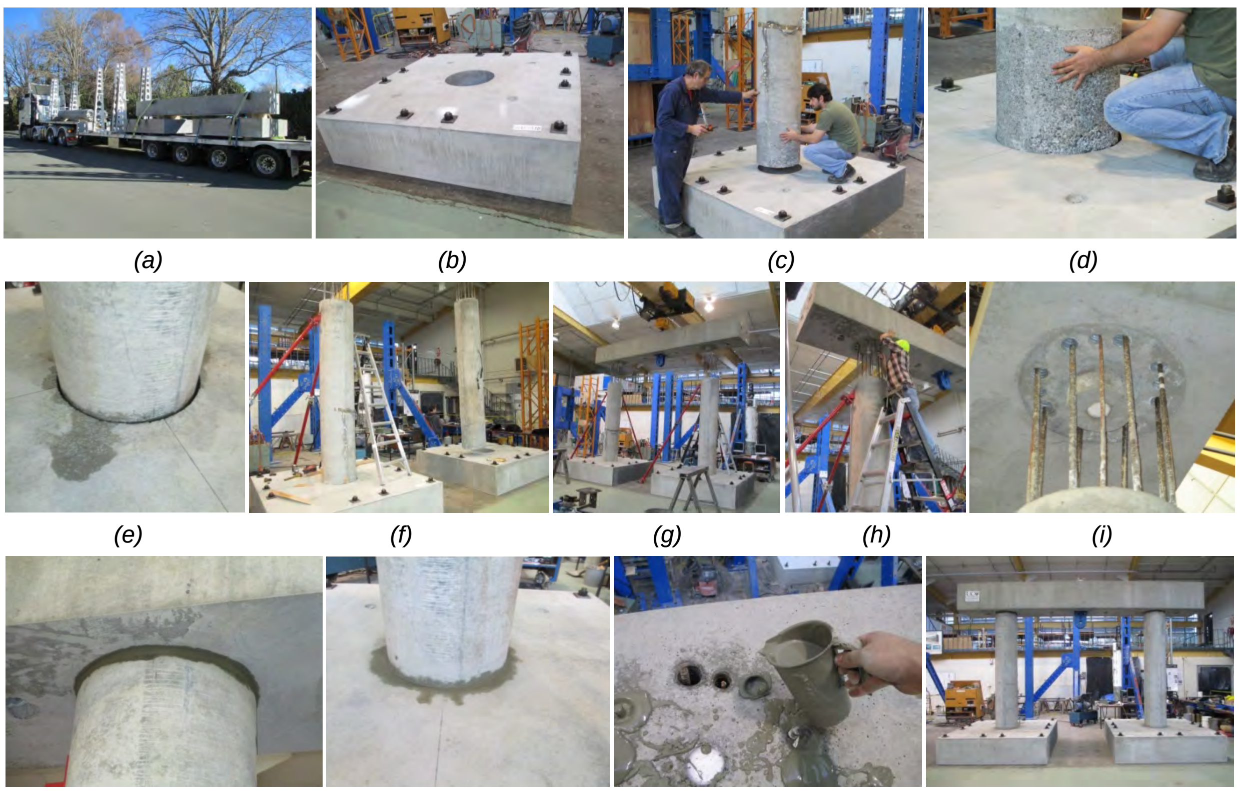


Figure 8. (Left) Internal actions for shear transfer in the grouted duct connection, (Middle) Primary stress transfer mechanism in the corrugated duct, (Right) Effect of un-bonded length on strain distribution of rebars at the interface

Assembly of ABC High Damage Bent

Figure 9 presents sequence of assembly and grouting for ABC High Damage bent.



(a) (b) (c) (d)

(e) (f) (g) (h) (i)

(j) (k) (l) (m)

Figure 9. (a) Precast elements delivered from the prefabrication yard, (b) Precast footing with socket, (c, d, and e) Sliding the precast column inside the footing socket, roughened surface of the column visible, (f) Temporary props installed for the columns, (g) Lowering the cap beam on the columns, (h and i) Aligning the starters bars inside the ducts, (j) Grouting bed under the cap beam, a layer of epoxy applied on the mortar to seal voids around the grouting bed, (k) Socket connections grouted, (l) Filling the ducts with grout, (m) Assembled bent, temporary props removed

Testing Arrangement

The testing is quasi-static cyclic loading with increasing drifts. Figure 10 shows the testing arrangement and loading history from ACI T1-01 (21) loading protocol for the specimen. Two hydraulic rams, each with 1000 kN capacity, were used to apply gravity and lateral loads to the bent. The gravity load was being held constant (to within approximately ±10kN) during testing.

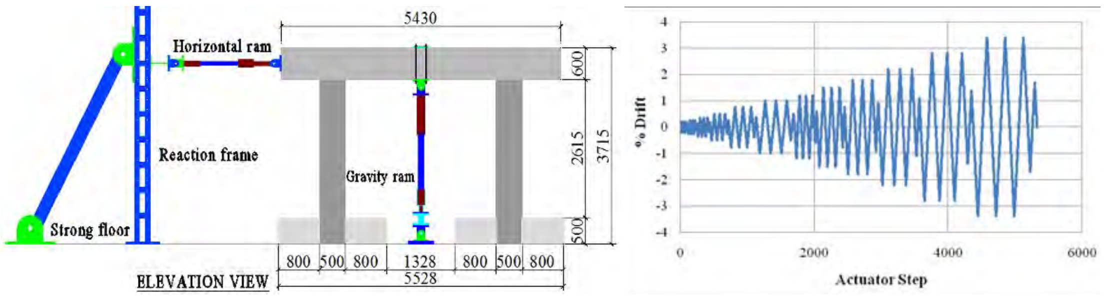


Figure 10. (Left) Test Setup, (Right) Displacement history for uni-directional quasi-static cyclic loading

Testing Results and Seismic Performance

Hairline cracking to the columns initiated during 0.2% drift cycles. For the member socket connections, the cracks were more distributed. Several big cracks were noticed along a height equal to diameter of the column (500 mm) from top of the footing. Cover concrete started spalling during 1.5% drift cycles. For the grouted duct connections, the grouting bed started deteriorating at 1.5% drift. There was minor spalling of cover concrete at 2.8% drift to a height of 200 mm below top of the column. The testing was stopped after 3.4% drift. It was obvious that the rupturing point for the rebars was greater than 3.4%. Using the displacement procedure outlined in Austroads Technical Report (22), the drifts for different performance levels are calculated, and are plotted on the force-displacement hysteresis (Figure 12). In Figure 11, measured crack widths in the plastic hinges for different performance levels along with the resulting ductility (μ) are presented. The moment capacities of all four connections are approximately the same.

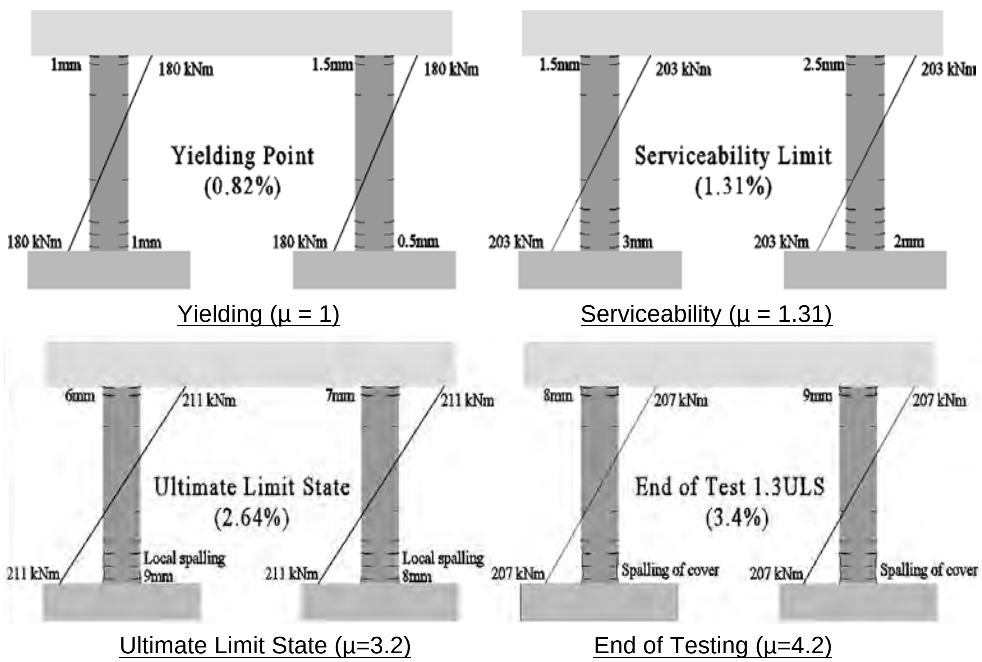


Figure 11. Moment distribution and measured crack widths in (mm) in the plastic hinges

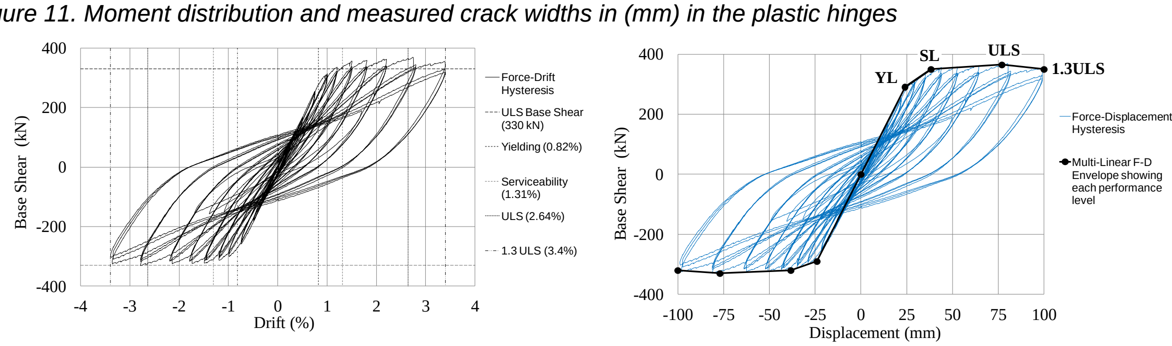


Figure 12. Hysteresis plots: (Left) base shear vs. drift (Right) Force-Displacement envelope

There was no damage to footings. There were few hairline cracks in the cap beam panel zones which were mainly caused by gravity loading. The system showed a very stable hysteresis. As expected, there were four plastic hinging zones in the columns.

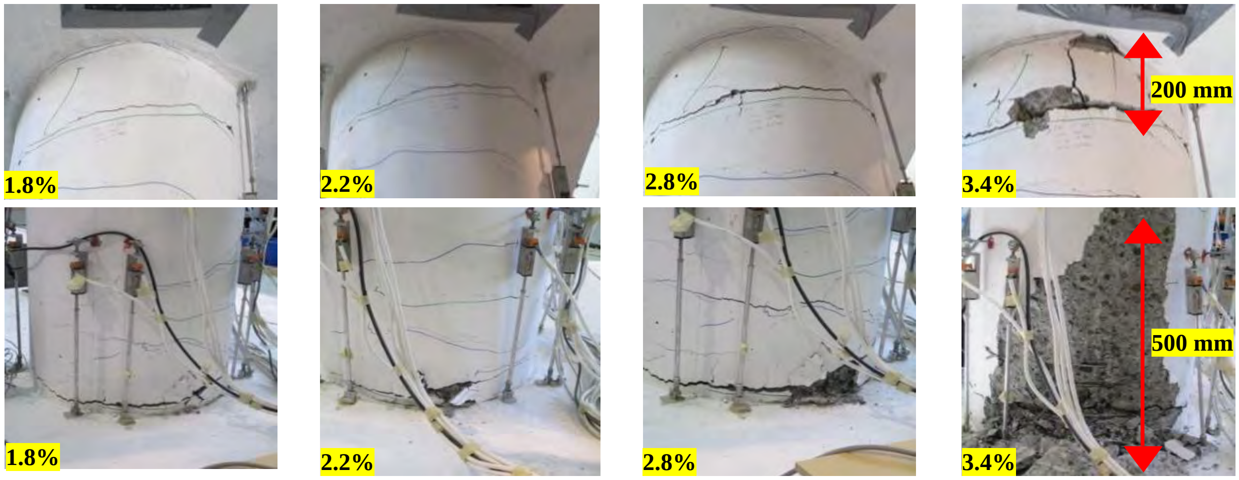


Figure 13. Damage in different drifts (Top row) Grouted duct connection (Bottom row) Member socket connection

From Figure 13, it is clear that the grouted duct connection has less concrete spalling compared to the member socket connection. One reason for this can be rocking mechanism of the grouted duct connection, the 100 mm un-bonded length of the starter bars at the interface have caused less spalling of the column cover concrete. Thus, it has resulted in less strength degradation and enhanced ductility.

2ND SPECIMEN: NON-EMULATIVE SOLUTION (ABC LOW DAMAGE)

The second specimen has the same dimensions as ABC High Damage. It uses a low-damage approach. The locations for potential plastic hinging are replaced by a combination of un-bonded post-tensioning with the external replaceable dissipaters. This solution is named Dissipative Controlled Rocking (DCR) or hybrid connection, and when combined with ABC concept can be referred as “ABC Low Damage” (Figure 14, Left and Middle). The post-tensioning provides self-centering, while the mild steel external dissipaters absorb seismic energy. The resultant hysteresis is commonly referred as “flag-shaped” (Figure 14, Right).

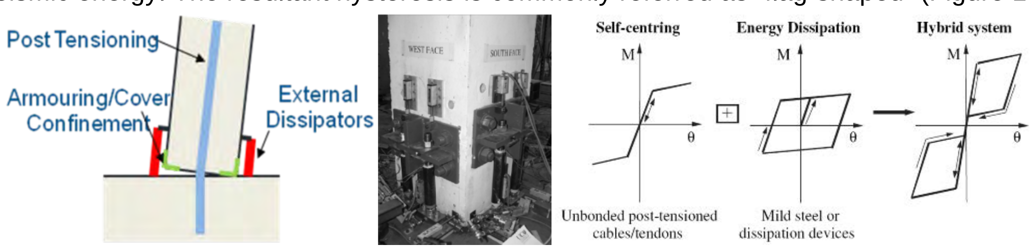


Figure 14. (Left and Middle) Typical DCR connection from Marriott (23), (Right) Concept of flag-shaped hysteresis

The cap beam was re-used. New concrete shear keys were cast for the cap beam and the footings. The rocking interfaces were armored with mild steel (Figure 15). Steel shells (10 mm thick) were used in the column ends for the outer confinement and to prevent concrete crushing when rocking. The shells are connected via welded studs to the column core and are placed around the cage when pouring the concrete at the prefabrication yard. The height of shells was 500 mm, which corresponds to a plastic hinge length of a ductile column for an equivalent cast-in-place construction according to NZS 3101 (15).

The design of a typical DCR connection can be done using NZCS PRESSS Design Handbook (17).The testing arrangement and loading protocol were identical to ABC High Damage. In the beginning three tests were done with different levels of initial post-tensioning. This aimed to investigate the response of the bent with un-bonded post-tensioning only. The levels of Initial Post-Tensioning (IPT) were selected to be 15%, 30%, and 45% of the yielding strength of the Macalloy bar. The initial post-tensioning (clamping force) controls the capacity of the connection, size of gap-opening at the rocking interface, and the re-centering ratio. In all three tests, the bent was taken up to 2.2% drift (slightly higher than its ULS design level) and did not suffer any damage. The columns remained intact without a hairline cracking.

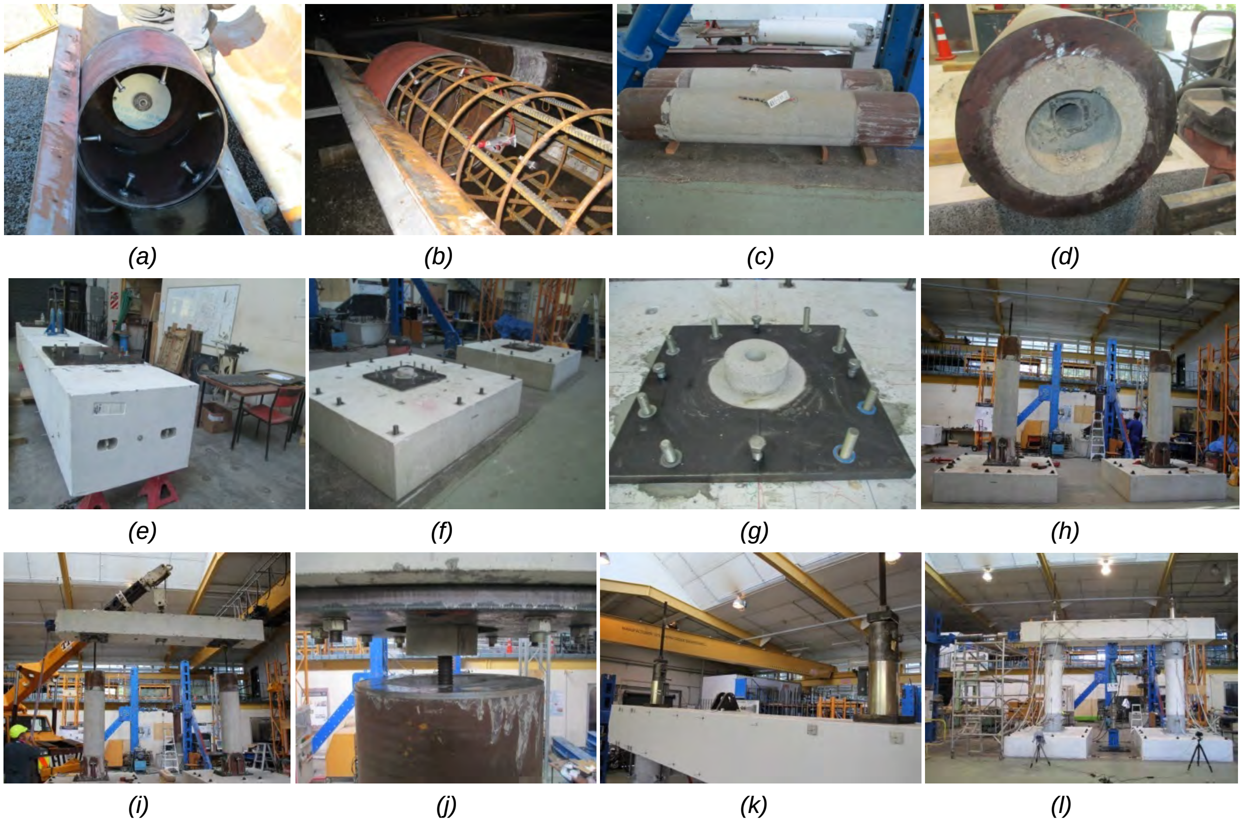


Figure 15. (a) Steel shell with welded studs , (b) Shell is placed in cage before concrete pouring, (c) Columns with armored ends, (d) Typical end view of the columns showing the recess for the shear key and central duct for the un-bonded post-tensioning Macalloy bar, (e) Cap beam with new shear keys and steel plates, (f) Footings with shear keys and steel plates, (g) Typical armoring of the rocking interface, close-up view showing shear key and steel plate, (h) Columns installed, Macalloy bar is running through the central duct in the column and is extending out, (i) Lowering the cap beam on the columns, (j) Aligning the cap beam shear key inside the column recess, (k) Macalloy bars are clamped on top of the cap beam, load cells and hydraulic jacks are placed, (l) Assembled Low Damage bent

There was 10 mm gap left between the cap beam shear key and the recess in the column. This was due to construction tolerances. It should be noted that in this case, the shear keys will not be grouted. They are simply provided to restrain any sliding in the connection. However, results from testing showed some twisting and sliding in the connections (Figure 17, top row). It was obvious that the internal round shear keys were not sufficient to prevent both sliding and twisting of the columns. Therefore, external steel shear keys were provided to solve the issue of twisting and sliding (Figure 16). All three testing with post-tensioning were repeated again using external shear keys. The size of gap openings at the top and bottom connections was almost identical. Figure 17 presents a comparison of the force-displacement plots with internal and external shear keys. For the external shear keys, it is evident that the force-displacement plot for testing with post-tensioning has been significantly improved (Figure 17, bottom row).

(a) (b) (c) (d)

Figure 16. (a) Bottom connection, external shear keys welded to base plate and column shell, steel brackets are used for mounting the dissipaters in the next phase of testing, (b) Shear key to prevent twisting of column during rocking, (c) Shear key to prevent sliding with a bit of angle cut on top to allow column rotation movement, (d) Top connection

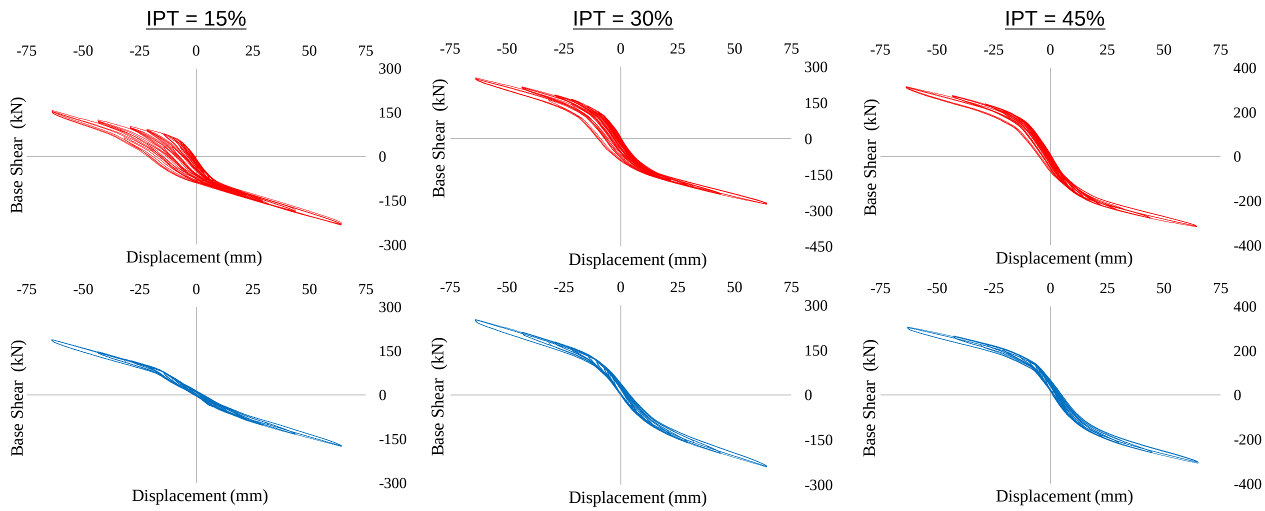
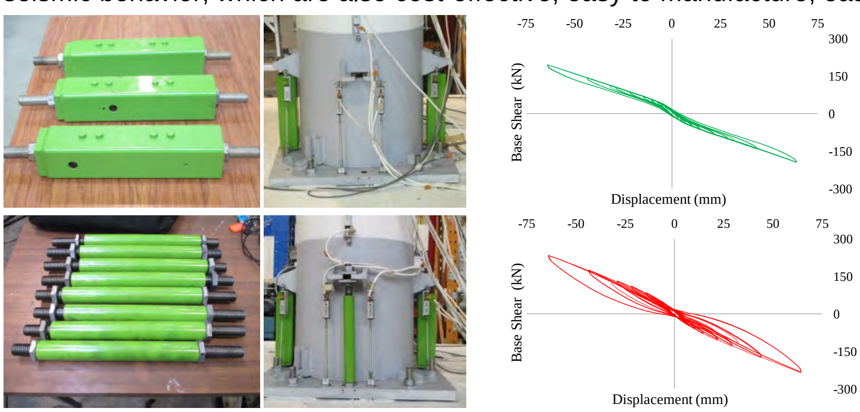


Figure 17. Testing results with: (Top row) Internal round concrete shear keys, (Bottom row) External steel shear keys

The response of post-tensioning from Figure 17 (bottom row) can be combined with mild steel dissipaters to generate a DCR connection with a flag-shape hysteresis. The dissipaters are mounted externally to the column as shown in Figure 18 (Middle). Both ends of the dissipaters are threaded. One end of the dissipater is winded to the threaded hole in the base plate, while the other end is tightened in the column bracket. Figure 18 shows two different arrangements of the external dissipaters in a typical DCR connection in ABC Low Damage bent. The dissipaters are mini plug and play devices with enhanced seismic behavior, which are also cost-effective, easy to manufacture, easy to install and replace.



<u>Arrangement 1:-</u> External mild steel dissipaters on two sides.

initial post-tensioning = 9% and no axial loads

<u>Arrangement 2:-</u> A combination of two types of external mild steel dissipaters around the DCR connection to enhance dissipation capacity.

initial post-tensioning = 9% and no axial loads

Figure 18. (Left) External dissipaters, (Middle) Installation in a DCR connection, (Right) Force-Displacement plots

The self-centering ratio and initial post-tensioning can be calculated using NZCS PRESSS Design Handbook (<u>17</u>) to match the capacity of ABC High Damage specimen at ULS .The bent was pushed up to 3% drift with no damage to the columns and zero residual displacement (Figure 18, Right). A comparison of force-displacement plots for the ABC High Damage and ABC Low Damage is given in Figure 19 (Left).

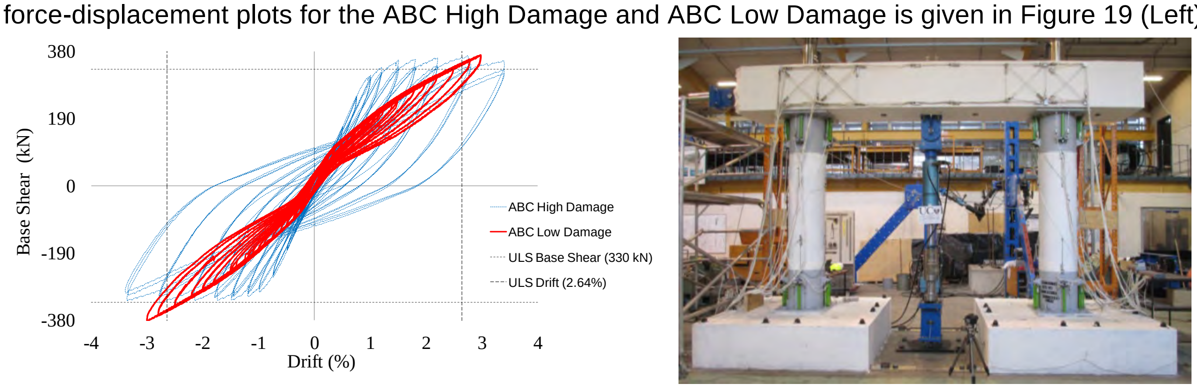


Figure 19. (Left) F-D plots for ABC High Damage vs. ABC Low Damage, (Right) No damage to bent after testing

CONCLUSIONS

The experimental testing of emulative connections for ABC showed promising results. The ABC High Damage bent achieved good strength and ductility levels by formation of four plastic hinges, similar to what can be expected from a cast-in-place construction. It has potential for significant time savings (precast cap beam, columns, and possibly footings). However, the cost of repairing and downtime will be an issue for ABC High Damage. Therefore, ABC Low Damage was developed as the evolution for a superior seismic performance which minimizes the cost of repair, in the same time eliminating downtime. ABC Low Damage has full re-centering capability. There was no damage to precast elements of the bent.

ACKNOWLEDGMENT

Authors would like to thank the New Zealand Ministry of Science and Innovation Natural Hazards Research Platform for supporting this research as part of the project ABCD, and technicians Gavin Keats and Russell McConchie for helping with the testing.

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Appendix 3

Controlled Damage Precast Connections for Accelerated Bridge Construction in Regions of High Seismicity

Sam White
Alessandro Palermo

Controlled Damage Precast Connections for Accelerated Bridge Construction in Regions of High Seismicity

2014 NZSEE Conference

S. White & A. Palermo

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch.

ABSTRACT: Bridge substructures are typically constructed using cast-in-place concrete components. During severe earthquake loading, these types of structures undergo inelastic deformation through the formation of plastic hinges. Although this type of approach has shown to be effective at achieving the base goal of ensuring life safety, there are some downsides relating to construction speed, quality and post-earthquake reparability.

Controlled Damage Connections are a type of precast connection featuring a combination of post-tensioning and energy dissipation components based on the principles of Dissipative Controlled Rocking (DCR) or Hybrid PRESSS. The use of precast components allows for accelerated bridge construction with improved construction quality.

The connections are detailed in a way that limits and constrains damage in bridge substructures during earthquake loading and minimises residual displacement of the bridge, meaning the bridge is more likely to be serviceable following an earthquake. Repair strategies are considered at the design stage allowing for rapid post-earthquake damage repair, minimising traffic disruption and repair costs.

At the University of Canterbury, half scale testing of two precast columns and footings featuring Controlled Damage Connections were undertaken as part of the New Zealand National Hazard Platform research programme titled Advanced Bridge Construction and Design (ABCD). The columns were subjected to displacement controlled biaxial loading. Following initial tests, the columns were repaired and re-tested to demonstrate the repair strategies and effectiveness. This paper presents findings of this experimental testing.

1 INTRODUCTION

This paper presents the design and testing of two half scale bridge piers featuring Controlled Damage Connections (CDCs). This connection type builds upon developments in Dissipative Controlled Rocking (DCR) or Hybrid PRESS connection types (Priestley, 1991, 1996; Palermo, 2004; Marriott, 2009; Pampanin et al,. 2010) and Accelerated Bridge Construction (Billington et al., 1999; 2004; Ralls et al., 2004; Stanton et al., 2005; Marsh et al., 2011)

CDCs offer the advantages associated with precast construction, notably increased construction speed and quality. However, they also limit damage during seismic events and provide simple and pre-planned cost-effective repair options. This is achieved through the provision of unbonded post-tensioned steel tendons or bars to limit residual drifts in the structure, combined with energy dissipation components which are easy to replace.

Two columns featuring Controlled Damage Connection types are discussed in this paper. The first is Column CDC (Figure 3) featuring the Controlled Damage (CD) Member Socket Connection (MSC) which is similar to the High Damage (HD) MSC presented in previous publications (Mashal, White & Palermo, 2013). The second is Column CDS (Figure 6) featuring the CD Coupled Bar Connection (CBC). This connection uses replaceable segments of longitudinal bar. Both connections were tested, repaired and retested to demonstrate the application and effectiveness of the repair strategy in each case.

This paper gives an overview of each connection type along with results of construction, testing and repair of the two test columns featuring CD connections.

2 PROTOTYPE STRUCTURE AND TESTING ARRANGEMENT

The prototype structure (Figure 1) is based on the Port Hills Overbridge in Christchurch, with a normal importance level, zone factor of 0.3, soil type C and no near field effects. A ductility of 3 was adopted for design with a design drift of 3%.

The test setup and loading protocol is shown in Figure 2. Each drift cycle consisted of a uniaxial push and pull in each direction followed by a clover shaped displacement path.

During testing, a 50mm post-tensioned bar was used to represent both the post-tensioning and gravity loads in the column.

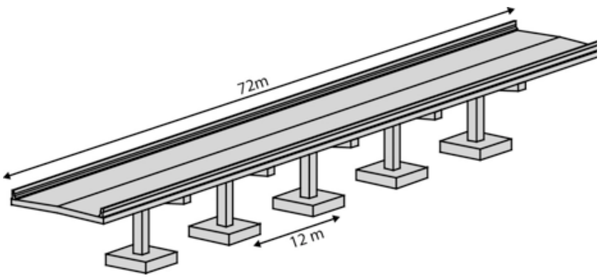

Figure 1a. Prototype bridge system

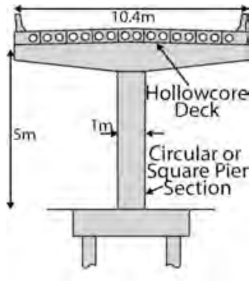

Figure 1b. Prototype transverse configuration

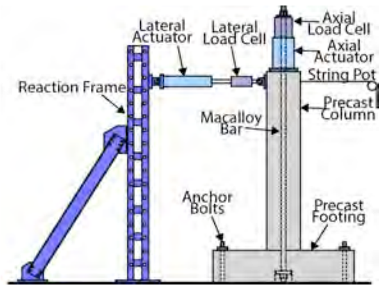

Figure 2a. Elevation of test setup

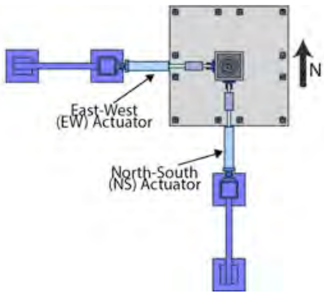

Figure 2b. Plan of test setup

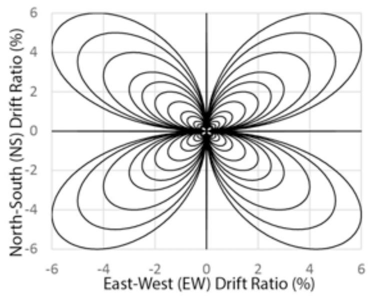

Figure 2c. Displacement Input

3 CONNECTION OVERVIEWS AND REPAIR STRATEGY

Both half scale columns had a section depth of 500mm and a total height of 3.2m. Square precast footings of 500mm depth and 2.1m length were used in both cases. Concrete with a strength of 50MPa was specified for both column with Grade 300 steel used for all components that are designed to yield as part of the energy dissipation system or provide armouring protection to the concrete. Grade 500 steel was specified for all other steel components. The design of both columns was based on the PRESSS design handbook adopting a re-centering ratio of 1.5 (Pampanin et al., 2010).

3.1 Controlled Damage Member Socket Connection

The Controlled Damage (CD) Member Socket Connection (MSC) (Figure 3) features a Member Socket Connection (Mashal, White & Palermo, 2013) with post-tensioning to limit residual drifts in the structure. Cover confinement limits spalling damage. Threaded anchors are cast into the precast components allowing for simple mounting of external dissipators for repair of the connection. The longitudinal bars were debonded over a length of 50mm at the connection interface, localising yielding of the bar and encouraging a rocking interface to form. The repair strategy is illustrated in Figure 4 and construction and repair photos are shown in Figure 5.

For repair of the connection, a novel dissipator design known as the Grooved Bar dissipator was used. This dissipator features a plain steel bar with grooves milled along the length of the dissipator, reducing the section area in the milled region to localise yielding in the bar. A steel tube is slid over the dissipator to prevent buckling of the dissipator under compressive loading without the need for filler material such as epoxy or grout as used in BRF style dissipators (Sarti, 2013).

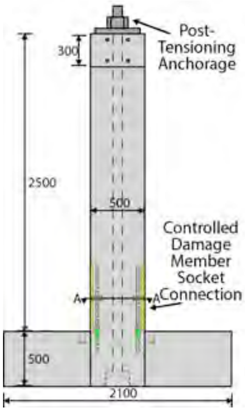

Figure 3a. Column CDC

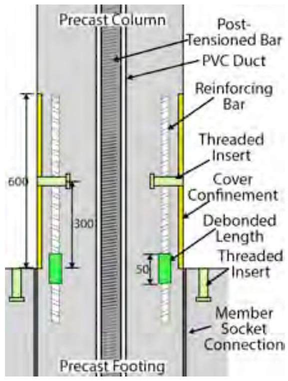

Figure 3b. Controlled Damage Member Socket Connection

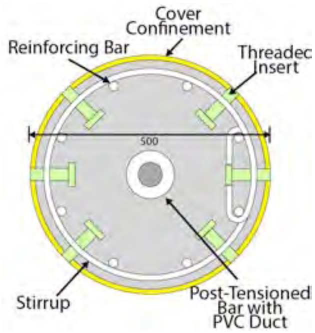

Figure 3c. Section A

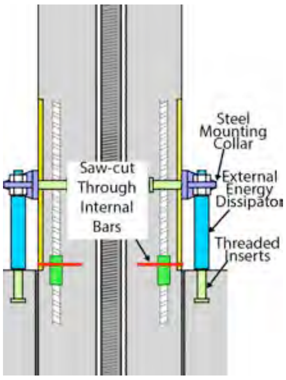

Figure 4a. Repair strategy for CDC

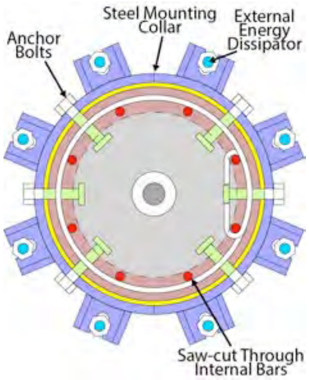

Figure 4b. Section A after repair

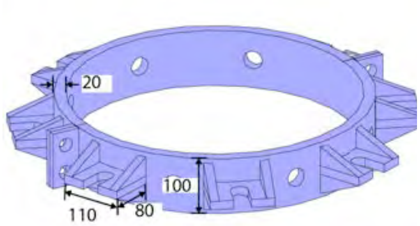

Figure 4c. Dissipator mounting collar

Figure 5a. Placement of column

Figure 5b. Application of FRP

Figure 5c. Column after repair

3.2 Controlled Damage Coupled Bar Connection

The second connection type is the Controlled Damage (CD) Coupled Bar Connection (CBC) (Figure 6). Replaceable segments of longitudinal bar are used, connected to permanent reinforcement using threaded bar couplers. The replaceable segments of bar are located in a recess in the precast column element which is filled with cast-in-place concrete or grout during construction. Steel armouring is used to protect the precast concrete core, meaning all damage is constrained to the cast-in-place material and replaceable dissipators. Figures 7, 8 and 9 illustrate the construction and repair methods.

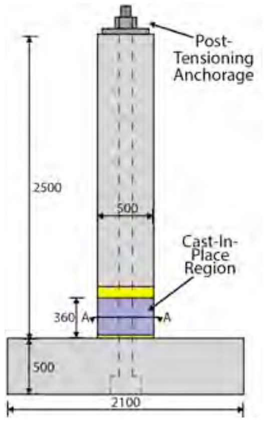

Figure 6a. Column CDS

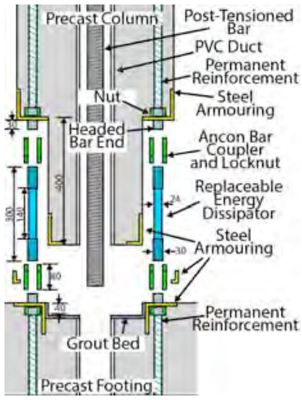

Figure 6b. Controlled Damage Coupled Bar Connection (exploded view)

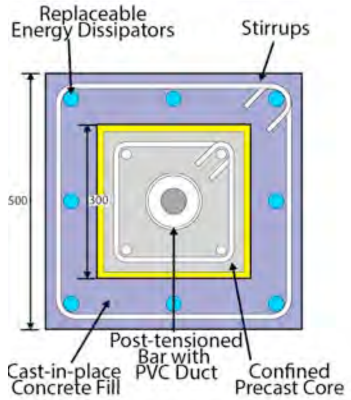

Figure 6c. Section A

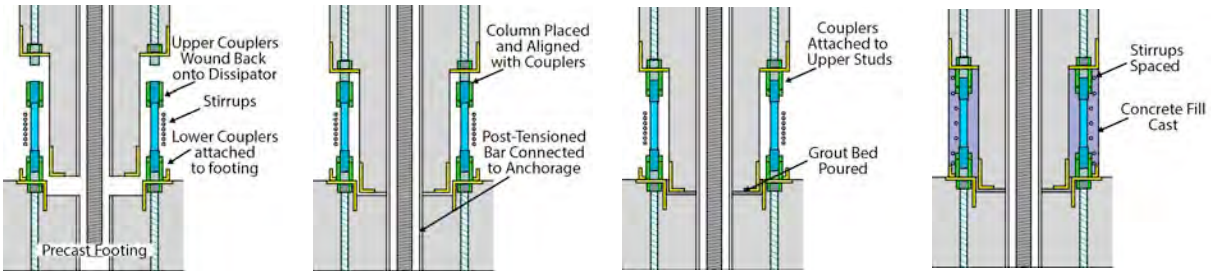

Figure 7. Controlled Damage Coupled Bar Connection assembly process

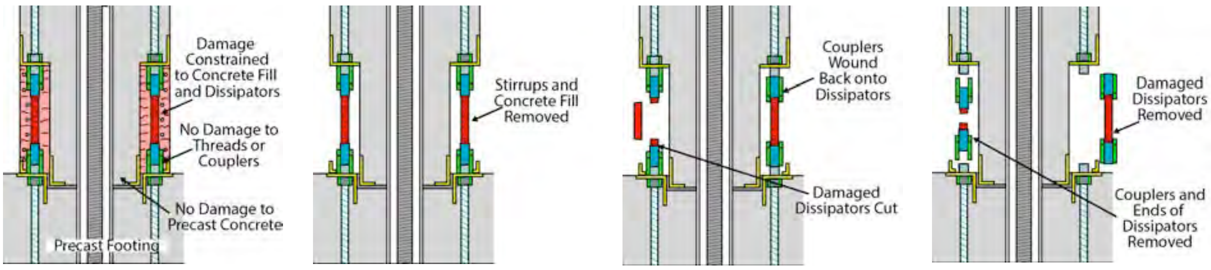

Figure 8. Controlled Damage Coupled Bar Connection repair process

Figure 9a. Dissipators and couplers

Figure 9b. Placement of column

Figure 9c. Construction complete

4 TESTING AND REPAIR

4.1 Controlled Damage Member Socket Connection

Figure 10 shows benchmark testing of Column CDC, Figure 11 shows the testing of the column after application of the repair strategy and Figure 12 gives the uniaxial force-drift behaviour observed in each test.

Figure 10. Column CDC benchmark testing

Figure 11. Testing of repair strategy

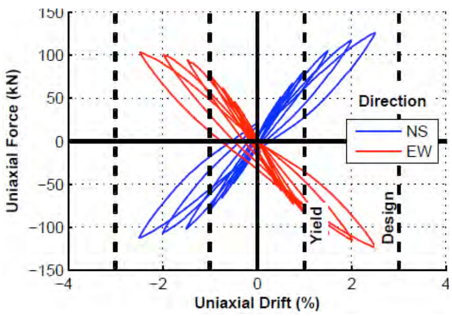

Figure 12a. Benchmark force-drift behaviour

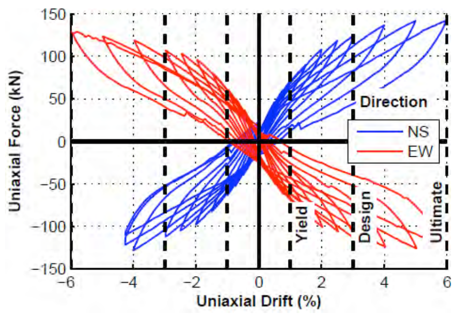

Figure 12b. Repaired force-drift behaviour

The results are summarised as follows:

- Good benchmark performance was observed with no spalling up to drifts of 3.25%.
- Post-drilled anchorages were used for connection of dissipators rather than threaded inserts which complicated the repair process and resulted in some undesired collar slip.
- Some pull-out of dissipators occurred, partly due to prior damage to the footing and the use of post-drilled anchorages.
- The column was subjected to drifts of up to 7.8% with no failure of the dissipators themselves.
- Despite the shortcomings in anchorage of the dissipators, good performance was seen in both the pre and post-repair connection with a clear flag shape visible in the hysteresis loops following repair.

4.2 Controlled Damage Coupled Bar Connection

Figure 13 shows benchmark testing of Column CDS, Figure 14 shows the testing of the column after application of the repair strategy and Figure 15 gives the uniaxial force-drift behaviour observed in each test.

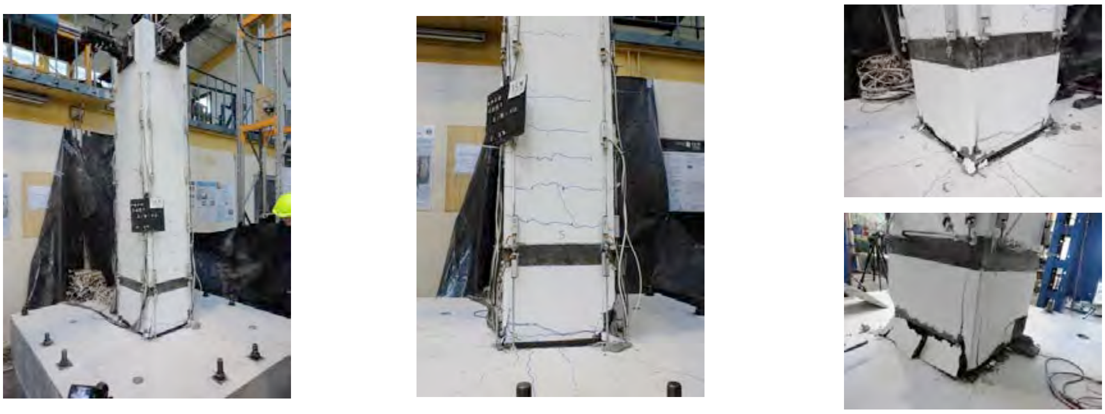

Figure 13. Column CDS benchmark testing

Figure 14. Testing of repair strategy

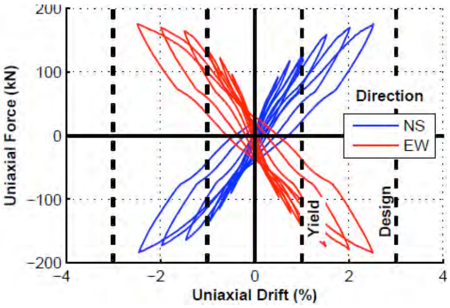

Figure 15a. Benchmark force-drift behaviour

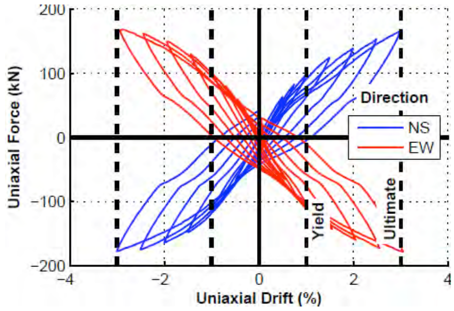

Figure 15b. Repaired force-drift behaviour

The results are summarised as follows:

- Benchmark testing of Column CDS showed good results although the flag shape was not as pronounced as in the previous CD tests. This is partly due to unintended bonding of the precast core to the footing which restrained the rocking behaviour of the joint and increased capacity and energy dissipation. This led to increased residual drifts in the structure however they were still considerably smaller than those of the HD tests.
- No slackness in the results was observed indicating good connection between replaceable dissipator, coupler and threaded stud with little slippage of the connection.

- After removal of cast-in-place fill and stirrups as part of the repair process, it was noticed that some buckling of the dissipators had occurred. The amount of buckling was limited but could be further reduced with an increase in the amount of buckling restraint in the form of stirrups, steel tubes over the dissipators or external cover confinement in the cast-in-place region.
- The damaged dissipators were removed and replacement Grooved Bar dissipators were installed. Some thread alignment challenges were faced during replacement but were overcome by swapping the location of replacement dissipators which did not offer significant delays to the repair process.
- Replacement stirrups were installed and fill material was cast, completing the repair of the connection. During casting, aggregate blockage of the fill tube was encountered and so grout was used in place of micro-concrete with no apparent effect on the performance of the column.
- During testing of the repair process, premature failure of the replacement dissipators occurred due to an identified detailing error. Previous tests have shown that with appropriate detailing, the dissipators could achieve larger strains without failure and so it is expected that the connection could have reached a higher level of ultimate drift.
- Otherwise, good performance with very similar results to the pre-repair testing indicating that the repair process was effective at reinstating both the strength and ductility capacity of the column.

5 RESULTS AND DISCUSSION

5.1 Comparison of Connections

Both connections showed promising results with flag shaped hysteresis loops occurring. Residual drifts were considerable smaller than those of the HD testing (Mashal, White & Palermo, 2013).

The two connection types demonstrate different approaches in the development and application of repair strategies. For the CD MSC, damaged energy dissipation components were severed and new components were installed on the exterior of the pier, offering an alternative energy dissipation system. This approach requires design for both internal and external dissipation systems but offers a much simpler repair process with no repair or replacement of concrete or grout required.

For the CBC, the repair approach involved replacement of the components of the energy dissipation system rather than installation of an alternative system. This approach offers a simpler design process where only one dissipation system needs to be considered and offers aesthetic advantages but requires a more involved repair process with removal and replacement of cast-in-place fill and stirrups. The repair process however is still significantly simpler than that of the HD or conventional monolithic systems where repair or replacement of reinforcing bars and cast-in-place concrete may be required along with difficulties associated with residual drifts of the structure.

6 CONCLUSIONS

The experimental testing of the Controlled Damage (CD) Member Socket Connection (MSC) and Coupled Bar Connection (CBC) through biaxial loading was presented in this paper. A repair strategy was applied to each connection type and the columns were re-tested to demonstrate the repair process and effectiveness. Assembly of both connections was straightforward and demonstrated the advantages of precast substructures in terms of speed and ease of assembly.

Despite some shortcomings in the construction and testing of the columns, both connection types showed promising results both in increasing construction speed and quality, and significantly reducing post-earthquake repair cost and downtime.

CD connections will generally have a higher initial construction cost than HD connections due to the inclusion of post-tensioning, armouring and replaceable energy dissipation components. This additional cost, however, may be balanced by the reduced repair cost should a significant earthquake event occur. Consideration of the full life cycle costs of the structure is required when comparing CD connections in order to account for all benefits associated with the system, rather than focusing only on initial construction cost. This includes using a reasonable discount factor when undertaking a benefit cost analysis to appropriately account for future benefits of the system.

Application of the repair strategy removes all uncertainty into the residual strength and ductility capacities of the connection, as all energy dissipation components of the connection are replaced. The relatively low cost of the repair strategies mean they can be implemented in any case where there is uncertainty in the level of damage of components and capacity of the substructure as a whole, avoiding the need for costly investigative procedures to be implemented.

Both connection types show good potential for use in bridge substructures. They offer the advantages of precast construction, notably increased speed and quality of construction, along with simple and cost effective repair. Further investigation into durability of the connections is being carried out at the University of Canterbury. Although Controlled Damage connections have higher initial construction costs, with further research and development and appropriate consideration of life cycle costs, it is expected that they offer a competitive alternative to conventional construction approaches while improving post-earthquake serviceability and repair options of bridge structures.

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Appendix 4

Investigation into PRESSS Bridge Piers in Highly A Seismic Region

Anton Kivell
Jamil Khan
David Hoffman

INVESTIGATION INTO PRESSS BRIDGE PIERS IN HIGHLY A SEISMIC REGION

Anton Kivell[1], Jamil Khan[2], David Hoffman[3]

ABSTRACT: *This paper outlines an investigation undertaken into the application of PRESSS technology for the bridges on the MacKays to Peka Peka Expressway (M2PP). This technology was seen as offering not only improved seismic performance, but also significant savings in construction time. While the benefits were clear, a number of risks were also identified with the implementation of such technology. A critical investigation was undertaken by the M2PP Alliance into PRESSS's viability and suitability for such a challenging project. The methodology and findings of this investigation are outlined in the following paper.*

KEYWORDS: Bridges, PRESSS, Seismic, Accelerated Construction

[1] Anton Kivell, Structural Engineer, Beca Ltd. Email: Anton.Kivell@beca.com
[2] Jamil Khan, Associate - Structural Engineering, Beca Ltd. Email: Jamil.Khan@beca.com
[3] David Hoffman, Fletcher Construction Ltd. Email: DavidHo@fcc.co.nz

1. INTRODUCTION

An investigation was undertaken into the viability of utilising Hybrid PRESSS technology to give the piers of the bridges rocking, re-centring characteristics on the MacKays to Peka Peka Expressway (M2PP). PRESSS[1] details for three of the bridges were developed. The Hybrid PRESSS system provides a number of attractive features around seismic performance and construction speed; however it also carries a number of risks and uncertainties due to the lack of real world application.

The Hybrid PRESSS system and its alternatives provide a novel approach to building structural bridge systems that can withstand extreme seismic and soil conditions, and be constructed considerably quicker than traditional construction. The location of the M2PP project presents the challenges of high seismicity and extensive deposits of liquefiable soil. These project circumstances have stimulated innovative thinking around seismic design and have produced concepts that differ from those traditionally used in New Zealand bridge design.

2. BACKGROUND

2.1. PROJECT

The MacKays to Peka Peka section of the SH1 - Wellington Northern Corridor Road of National Significance (RoNS) passes through the Kāpiti Region, 60km north of Wellington, New Zealand and will be built to expressway standards i.e. four-lane, median divided carriageway. The project will be delivered by a consortium (the MacKays to Peka Peka Expressway Alliance) formed of NZ Transport Agency (NZTA), Fletcher Construction, Higgins Contractors and Beca.

The seven RoNS projects are based around New Zealand's five largest population centres. The focus is on moving people and freight between and within these centres more safely and efficiently. The RoNS are 'lead infrastructure' projects – that is, they enable economic growth rather than simply responding to it. The purpose of the SH1 – Wellington Northern Corridor improvements is to improve the efficiency of State Highway traffic to and from Wellington CBD, port and airport and improve the resilience of the primary link to Wellington.

One of the key post-disaster roles of the Expressway is to provide a modern and reliable crossing over the Waikanae River. In addition to the 182m long Waikanae River crossing, 16 other bridges are to be constructed, crossing local roads and waterways. The challenges associated with this project are considerable and unique compared to many other bridge projects in New Zealand. The high seismicity of the Kāpiti coast, combined with a ULS design return period of 2,500 years, produces design peak ground accelerations of up to 0.98g. To compound this, most of the route is on poor soils which are expected to liquefy under approximately a 1/250 year event to depths of 6-12m. Combined, these factors have forced innovation, and adoption of advanced design procedures and technology, to overcome the extreme demands placed on these structures.

The project has progressed at a fast pace, with a strong emphasis placed on innovation and construction efficiency. The team found that early construction of bridges crossing key waterways allows for efficient transportation of fill material along the length of the project and a shortening of the overall construction schedule. The designers' response was to investigate alternatives that allowed for rapid bridge erection. One of the potential methods of reducing construction time was through jointed pre-cast construction using post-tensioned system for construction of piers and cross-heads.

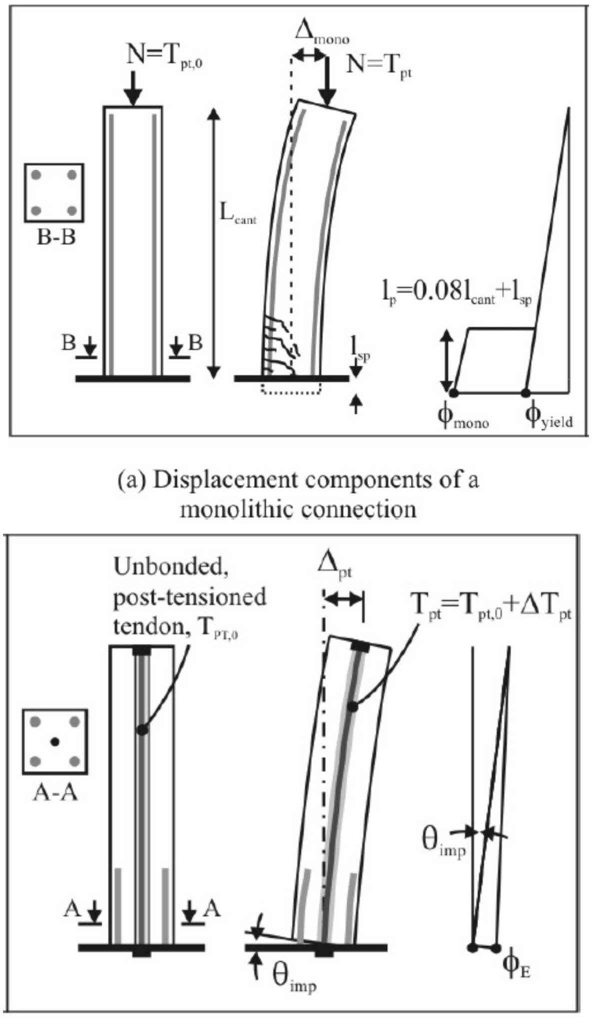
2.2. HYBRID ROCKING POST-TENSIONED STRUCTURES

Hybrid rocking post-tensioned structures provide a structural solution that reduces damage to the structure in seismic events. This technology substitutes formation of plastic hinges in ductile regions of the structure for jointed rocking connections between precast elements as the seismic system. Provided these rocking connections are detailed correctly, damage at the rocking interface can be limited with avoidance of extensive damage associated with the formation of traditional reinforced concrete plastic hinges.

Furthermore, installation of post-tensioning introduces a re-centring characteristic to the structure. Following a design level seismic event the structure can be designed so that minimal residual drifts would be expected, allowing for an immediate return to service following a ULS event (after inspection). The global structural behaviour is very similar between the traditional and rocking systems, with similar force displacement characteristic displayed by both. The primary difference being that a traditional plastic hinge is replaced by a rocking interface. The system shares similar internal actions to a traditional plastic hinge, however some alteration to standard theory is required to account for the opening of the jointed connection. Figure 1 describes the monolithic beam analogy which allows for calculation of

[1] PRESSS – Precast Seismic Structural System

concrete strain at the rocking interface where strain continuity is lost.



(a) Displacement components of a monolithic connection

(b) Displacement components of a post-tensioned connection

Figure 1: Comparison between tradition pier (a) and rocking piers (b). (Monolithic beam analogy – modified from Marriot, 2009)

2.3. History

Development of rocking post-tensioned structures started in the 1990's through a large scale research project at the University of California, Berkley [1]. This focused on the implementation of post-tensioned rocking joints in buildings and was known as Precast Seismic Structural System or PRESSS. This system had the goal of minimising damage to the structure in design level earthquakes. Over the following two decades extensive research into the design and performance of these connections, resulting in their inclusion in the New Zealand concrete design standard, NZS3101:2006 [2], and development of the PRESSS Design handbook [3] by the New Zealand Concrete Society. This was accompanied by the construction of the first multi-storey PRESSS building in 2010 in Wellington, New Zealand. Since then, a number of PRESSS buildings have been constructed around New Zealand with a strong demand for 'low damage' solutions arising from the Canterbury earthquakes and the subsequent Christchurch rebuild.

In 2009, Dion Marriott completed his PhD Thesis [4] which investigated the design a rocking bridge piers amongst other rocking post-tensioned elements. The investigation included a case study which compared the performance of differing systems and compared the expected losses following a design level earthquake. Numerical analysis of rocking post-tensioned bridge piers has showed their viability and shown a high level of performance. However, no examples of rocking post-tensioned bridges designed for seismic applications are known to be currently in existence.

3. CONCEPT

The concept of a rocking post-tensioned pier is described in Figure 2 which shows a cantilever rocking bridge pier under seismic loading. An overturning moment is produced at the foundation joint by the seismic load applied at the top of the pier (V^*_{EQ}). This overturning moment is resisted by the moment couple developed between the concrete compression block (C_C), and the post-tensioned tendon (T_{PT}) and the dissipative steel (T_S). Shear force is resisted by shear keys (V_n).

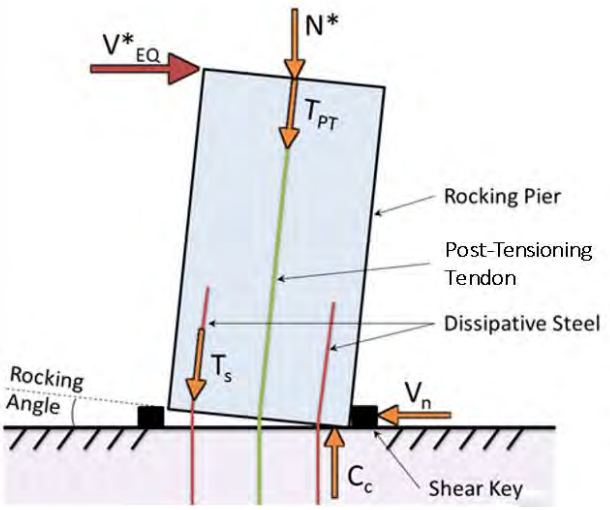


Figure 2: Diagram of Rocking Post-Tensioned Pier.

Re-centring is an important attribute of the rocking post-tension system to minimise residual displacement. For the pier to display re-centring behaviour, the restoring moment produced by the combination of the axial load (M_N) and the post-tensioning tendon (M_{PT}), must exceed the moment developed by the dissipative steel (M_{ms}), as this steel must yield in compression in the re-centring phase. The ratio between the re-centring moment and the dissipative moment defines λ, the re-centring ratio as defined below.

$$\lambda = \frac{M_{PT} + M_N}{M_{ms}}$$

To ensure recentring, the re-centring ratio is required to exceed the overstrength capacity of the dissipative steel moment to account for the increased steel strength that can be developed at large strains. Hence, for Grade500E steel $\lambda > 1.35$. A re-centring ratio of 1.5 is suggested in the PRESSS Design Handbook [3] to ensure re-centring. Hybrid PRESSS structures display a distinctive flag shaped hysteresis loop due to the combination of the elastic recentring component induced by the post-tensioning, and the yielding mild steel component. The enhanced elastic component results in diminished damping when compared to a traditional reinforced concrete detail.

4. DESIGN

Design procedures of rocking post-tension systems are provided in the PRESSS Design Handbook. These procedures share many similarities with a traditional section analysis with minor alterations to account for the loss of strain continuity at the rocking interface. These design procedures are based around a displacement based design philosophy and are consistent with the current draft of the NZTA Bridge Manual (Appendix A - 23/2/13). Appendix A provides a series of material strain limits to be used for dissipative steel (G500E), concrete and post-tensioning tendons at 'Damage control limit state' (ULS equivalent). However, Appendix A does not provide material strain limits for the Major Earthquake Event. Marriott [4] presents a set of Maximum Credible Event (MCE) strain limits, which were used for the MCE design checks.

Structural modelling and design was undertaken using a combination of the software package LPILE for foundation behaviour, and Microsoft Excel. This allowed for inclusion of foundation deformation in the Displacement Based Design process which was undertaken in a spreadsheet. Analysis and design of the rocking section was also undertaken in Microsoft Excel with the damping and re-centrering ratios checked and adjustments made to refine the design.

This methodology does not differ from a normal displacement based design. The only consideration required is through the reduced damping provided by the Hybrid PRESSS connection due to the re-centreing characteristics of the flagged shaped hysteresis.

5. ADVANTAGES

There are many advantages associated with a rocking post-tensioned structural system, the most influential of these are outlined below.

5.1. Seismic Performance

The rocking post-tensioned structural system was developed with the main objectives of limiting structural damage and limiting residual drifts. Marriot[4] demonstrated that the reduced structural damage resulted in reduced repair costs, along with greatly reduced down time. The ability for the bridges to sustain limited damage and re-centre following a design level earthquake means the structure should only require a brief inspection before it can return into service following a ULS earthquake. Contrary to this, a traditional structure has a significantly greater potential of coming out of a design level earthquake with a permanent displacement. The consequences of this range from impaired service, to the requirement for demolition due to excessive residual displacements.

These factors make the rocking post-tensioned system superior over traditional construction in the level of seismic performance it is able to achieve. The ability for post-tensioned rocking bridges to return to service almost immediately after a major seismic event is critical in their lifeline functionality requirements.

5.2. Construction

Use of precast elements and connection of these elements using post-tensioning greatly reduces erection times of the substructure which can be associated with significant cost savings in the construction phase. This increased construction speed is due to the ability for precast elements to develop the majority of their moment capacity immediately after post-tensioning. As these piers are designed for extreme seismic events, the moment demand on them under gravity load is expected to be minimal when compared to that produced by the post-tensioning allowing for immediate progression of construction following placement and tensioning of tendons.

Inclusion of post-tensioning steel centrally to the element allows for a reduction in dissipative reinforcement in the order of 50-60%. Traditional designs pose a high level of difficulty in construction due to joint congestion. Reduction of this steel allows further cost savings through reduced construction time.

Precasting of elements means that they can be fabricated in controlled conditions, to higher tolerances. It also reduces the dependence on weather conditions when placing steel and pouring concrete allowing for less contingency for weather in programming, and safer working conditions.

5.3. Durability

Utilisation of post-tensioning means that following a major seismic event, any minor cracks that may have formed in the piers during shaking (which remain elastic) will be closed following the event. Furthermore, the rocking interface is designed to close following shaking, providing an equivalent level of protection to reinforcement as prior to the event.

Traditional plastic hinge regions would be expected to be heavily cracked following a design level event. If not repaired quickly and correctly, ingress of external corrosive agents is likely to result in deterioration of the plastic hinge region, compromising the integrity of the most important regions of the seismic system.

5.4. Safety

Prefabrication of elements off site or on the ground eliminates safety issues associated with working at heights when placing steel and pouring concrete. It also limits exposure of the constructors to harsh environmental conditions. Time spent working at heights in erection will be greatly reduced with fewer people required to erect the substructure, working over a shorter period of time.

6. DISADVANTAGES AND RISKS

Some risks and disadvantages are associated with the rocking post-tensioned structural system, these are outlined below.

6.1. Seismic Performance

A rocking post-tensioned structural system will have diminished damping when compared to a monolithic equivalent due to its re-centring characteristics. This could be negated through the inclusion of additional damping devices. These would carry additional costs and are not used commonly at the current time.

6.2. Innovation

We understand that the M2PP bridges would be the first bridges in the world to be constructed using PRESSS technology. As a result, in both design and construction, it would be expected that there would be more unforeseen issues relating to constructability.

One PRESSS building was built in Christchurch at the time of the Christchurch earthquakes. This was the Southern Cross Hospital Endoscopy building. Minor damage was observed to this building, mostly around detailing of the interaction between structural and non-structural elements and was considered to have met the level of service expected under a design level earthquake [5]. Other than this building, no PRESSS buildings have been subjected to intense seismic loading outside of the laboratory.

Careful consideration needs to be given in design as to how the rocking post-tensioned system would behave in a real earthquake, and what the critical aspects that may inhibit its robust performance are. Unforeseen issues can be exposed by earthquakes where structural behaviour is not as expected in design modelling.

6.3. Construction

While precast elements can be fabricated with improved precision and quality, there is an increased potential for damage to them when they are being transported to site. Manoeuvring of such elements will require larger cranes which may require increased temporary works to provide a working platform for them. Precasting of elements does not provide for as greater tolerance when connecting structural elements and additional thought will have to be given to the constructability of interconnecting elements as clashes would be difficult to be corrected on site.

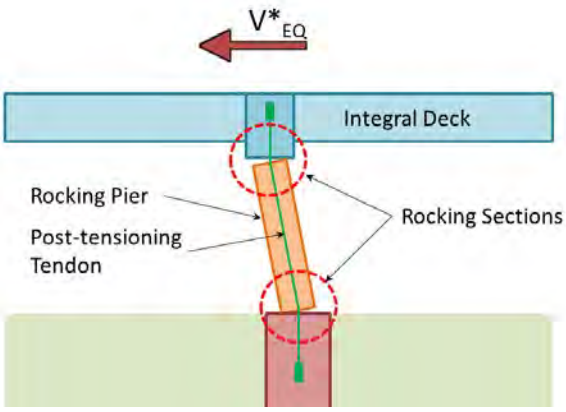
A general consideration for post-tensioning work is the premium that is paid for installation due to the requirement for expert installation of the post-tensioning. However, such specialists were available within the M2PP Alliance and it was anticipated that the costs for installation would be significantly less than if an external subcontractor was used.

6.4. Durability

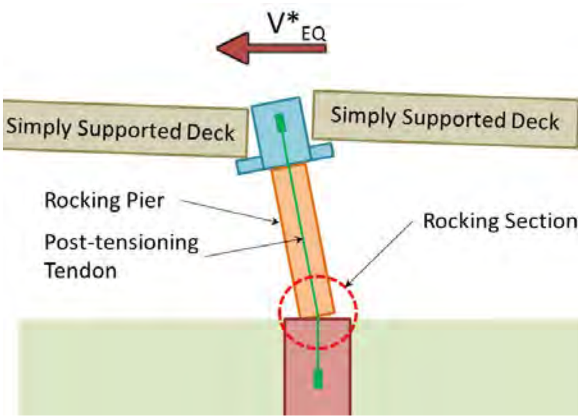
Careful consideration will need to be given around the durability of the rocking elements. It is essential that the post-tensioning tendons are protected from corrosion, and that elements are suitability protected from the external environment. Post-tensioning tendons must also be debonded to allow for distribution of tendon elongation along the full length of the tendon. Without the alkaline environment of concrete surrounding the tendons alternative methods of protection need to be considered. Similar concerns arise around the dissipative steel at the rocking interface where it is debonded.

7. Concept Investigation

Three bridges, Waikanae River Bridge, Wharemauku Stream Bridge and Kāpiti Road Overpass, were selected for concept investigation. These broadly covered the range of structural arrangements used in the project. A developed design of the seismic system was undertaken utilising rocking post-tensioned structural systems. The general design arrangements can be divided into two cases for integral bridges and simply supported bridges. The location of rocking sections in these bridge types are shown in Figure 3. Integral bridges will form rocking mechanisms at the top and bottom of the piers under lateral loading in the longitudinal and transverse directions, as shown in Figure 3(a). The simply supported bridges will have a single rocking section at the base of the piers in the longitudinal direction, with the piers acting as cantilevers as shown in Figure 3(b), but will behave like the integral bridges in the transverse direction with rocking sections at the top and bottom of the piers behaving as a frame.



(a)



(b)

Figure 3: (a) Integral Bridge Rocking Arrangement, (b) Simply Supported Bridge Rocking Arrangement

7.1. Items Investigated

The M2PP Alliance team identified a number of critical items that required investigation to determine the effect that they would have on the viability of the rocking post-tensioned concept. These items were;

1. Will rocking be initiated prior to hinging of the piles at depth under liquefied conditions?
2. How does a rocking section compare to a monolithic connection with regard to size, strength, ductility and plastic rotation capacity?
3. How can durability of steel elements be achieved at the rocking interface?

The findings from these investigations are outlined below.

7.1.1. Will rocking be initiated prior to hinging of the piles at depth under liquefied conditions?

To investigate the location of inelastic behaviour, models of the bridges, including piles supported by soil springs, were created for both the liquefied and non-liquefied conditions. Design capacities were determined for the pier sections and then their overstrength capacity applied at rocking locations. From these it was determined if there was potential for hinging at depth. This could take the form of; hinging at depth prior to formation of hinges in the rocking structure, or hinging at depth at overstrength of the rocking structure.

At Kāpiti Road, an increase in pile size to 2.4m diameter was required to avoid hinging in the piles prior to rocking. At the Waikanae River Bridge and Wharemaku Stream Bridge the piers were found to rock as desired at yield and overstrength.

The formation of plastic hinges in the piles was not critical for the ULS seismic performance. These hinges would form under traditional monolithic pier conditions and do not compromise the stability or seismic performance of the structure. However, they will not provide dependable re-centring of the structure negating some of the benefits of rocking post-tensioned piers. This was not considered to be critical as the ductility requirements on the pile hinges are significantly lower than those of the pier hinges, decreasing the likelihood of appreciable residual displacements. While hinging at depth is not as desirable as at the rocking interface, its effects were no greater than what would occur if a monolithic pier was implemented and predominantly only occur under fully liquefied conditions (lower bound soil springs).

7.1.2. How does a rocking section compare to a monolithic connection with regard to size, strength, ductility and plastic rotation capacity?

There were concerns around joint congestion in the monolithic design. Inclusion of post-tensioning tendons allows an equivalent section capacity to be achieved by a rocking section, meaning that the section size does not need to increase from a monolithic equivalent. This in turn allows for reduced reinforcing steel to be used around the perimeter of the rocking section.

In the New Zealand concrete code, NZS3101:2006 [2], the maximum allowable plastic rotation is ~0.04rads for a fully ductile plastic hinge. From rocking sections analysis a plastic rotation of 0.035rads was able to be achieved at ULS strain limits [6], with ~0.055rads at MCE strain limits. These rotations are in line with those described by Marriott [4] where he used 3.56% (0.0356rads) as ULS maximum rotation. The inclusion of tendons increases the axial loading on the section requiring increased concrete compression, which was found to limit the rotation capacity. This was alleviated somewhat through use of higher strength concrete in the design of the rocking piers than was used in the traditional design.

While the ductility capacity of the pier may be slightly reduced, the overall structural ductility is highly influenced by pile flexibility meaning that the reduced pier ductility has minimal impact on the overall structural ductility. However, as mentioned earlier, the damping provided by the inelastic joint is reduced from a monolithic equivalent.

7.1.3. How can durability of steel elements be achieved at the rocking interface?

Durability over 100 years of service requires careful consideration and attention to detail around the rocking interface. The three elements which were of primary concern were: the post-tension tendons, reinforcing steel, and the steel armouring around the rocking perimeter.

The approach to protecting the internal post-tensioning tendons was to grout greased tendons within a High Density Plastic (HDP) tube after they had been tensioned. This HDP tube then sat unbounded within a Drossbach duct. This solution provides two layers of protection to the tendon, the grout and the HDP tube. It is expected that the HDP tube will be run the full length of the tendon and that it will be able to sustain the deformations that it will be subjected to under rocking.

Reinforcing steel was housed within the concrete section. De-bonding of this reinforcement was achieved by application of Denso tape to the required debonded length. The reinforcing bar will be protected by cover concrete, or high strength grout when within a Drossbach duct. Use of galvanising of the reinforcing steel along the debonded length in conjunction could also have been used for additional surety.

Steel armouring was provided around the rocking perimeter to help distribute peak stresses associated with rocking impact. This steel will be exposed to the external environment and so stainless steel was adopted.

7.2. Conceptual Section

From the investigation which has been undertaken the following design has been produced for the structural section at the rocking interface. An example from the Kāpiti Road Bridge is shown in Figure 4. To provide adequate capacity in the longitudinal direction without excessive capacity in the transverse direction, the section is reduced locally around the rocking interface. The traditional reinforced concrete design of the Kāpiti Road Bridge pier used 60 DH40 bars. This was reduced to 26-DH40 bars and 120 15.2mm post-tensioning tendons housed in 4 central ducts. The rocking perimeter had stainless-steel armouring shoes, (~150x10EA) which were be cast into the pier around its perimeter. The rocking surface of the pile was armoured with stainless steel plate to minimise contact damage. The DH40 bars were de-bonded a length of 200mm at the rocking interface. Tendons were greased and stressed in high density plastic tubes then grouted. These tubes were in Drossbach ducts. This ensured durability and de-bonding along the entire length of the tendon. 65MPa concrete was be used for the pier.

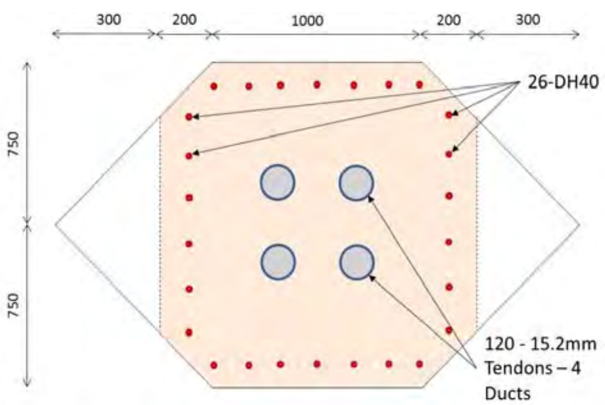


Figure 4: Bridge Rocking Section Design (Example from Kapiti Road Underbridge)

8. COMPARISON OF ROCKING AND TRADITIONAL DESIGN

8.1. Costs

Costing was undertaken for the Waikanae River Bridge, the largest bridge on the MacKays to Peka Peka Expressway project. The comparison design was undertaken based on an integral 1825 Super-T deck with a total length of 182m consisting of five spans. The bridge carries two expressway lanes of traffic in each direction and includes a pedestrian footpath/cycleway on one side of it. A cost comparison was undertaken between the PRESSS design and a conventional monolithic design. This showed savings in temporary works, precasting, erection time and site costs. A small premium was paid for the materials but this was not significant, and increased costs came through increased crane size and transportation of large precast elements.

It was initially concluded that there was some cost advantage in using the PRESSS system over a traditional monolithic design. However further investigation was undertaken.

8.2. Further Investigation

An investigation into an alternative, more conventional precast method was also undertaken to see if comparable erection times could be achieved without the use of post-tensioning. This concept involved pouring a cast in-situ stitch at the top of the pile whilst the precast pier was supported in the void at the top of the pile, or plunging the precast pier reinforcement into fresh concrete at the top of the pile. The pier reinforcement was too closely spaced to use ducts cast into the top of the pier.

From this investigation it was concluded that comparable erection times for both methods of pier construction could be achieved. The reliance on large early strength gains of the grout used in the trumpet anchorages to allow stressing, meant that similar curing times were required for the in-situ stitch and the post-tensioned option.

A revision of the bridge form across the project meant a non-seismic connection was now required at a number of bridges where a ‘hammer-head’ pier-crosshead arrangement had been adopted. To reduce joint congestions and increase erection speed a solution using threaded stress bars was developed, as shown in Figure 5. The system involved steel anchorage plates to be cast into the pier–column member. Upon erection, threaded stress bars were screwed into these plates, the crosshead was then lowered over them with the stress bars passing through oversize ducts, and then stressing was performed. This still allowed for rapid erection of the crossheads but reduced the demand on precasting and transportation, proving to be the most cost effective solution.

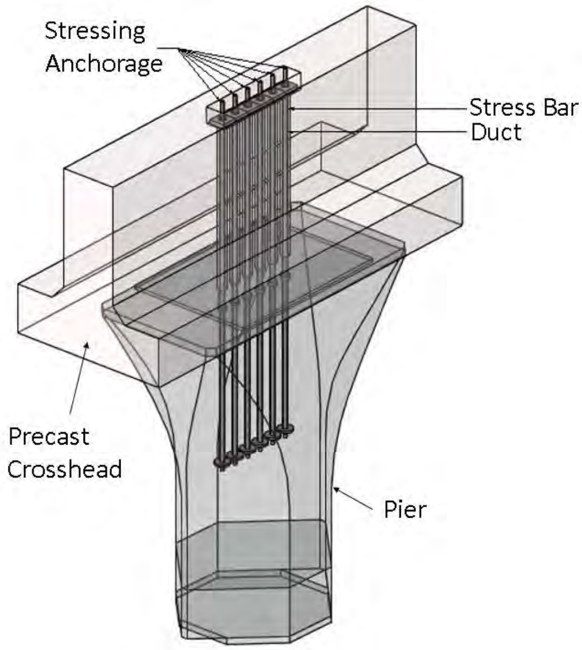


Figure 5: Final pier-crosshead connection arrangement

9. OUTCOME

A number of factors influenced the final decision on the application of hybrid PRESSS technology to the M2PP expressway.

While savings were predicted in erection time, construction of the bridge sub and super-structure was later found to not govern the critical path of the bridges. Extensive ground improvement at the abutments and construction of the MSE block approaches had the greatest influence on programme. With the primary savings stemming from reduced preliminary and general costs, having the bridge structure off the project critical path meant that these saving in construction time would not be realised..

There were also constraints on the availability of nearby precasting facilities, with most of the availability being used for construction of the precast bridge deck beams. Furthermore, to vertically cast the piers would require 7-9m of crane clearance so any precast pier option would require a construction of a specialist casting facility.

These factors combined with those covered earlier in this paper meant that the Hybrid PRESSS option was not adopted for the M2PP Expressway project.

However, advantages were seen in the ease of construction of the precast crosshead with a post-tensioned connection between the crosshead and pier using stress bar and this was adopted in the final design.

10. CONCLUSION

An investigation into the viability of a rocking post-tensioned seismic structural system (Hybrid PRESSS) was undertaken for application for the bridges on the Mackays to Peka Peka Expressway. Designs for several bridges were undertaken with details developed to address a number of key considerations. These designs provide a number of beneficial characteristics including their recentring ability after a major seismic event and their speed of erection.

The total cost of the system materials was approximately neutral, with savings achieved in construction time. With these savings, the Hybrid PRESSS system was pushed forward as being of real benefit to the project and time and effort put into developing it further. It was concluded that the Hybrid PRESSS system did provide a viable and competitive structural system.

However, due to project specific factors, the Hybrid PRESSS pier option was not seen as offering the best solution for the bridges on the M2PP expressway project, primarily due to the capacity of local precasting facilities and because the above ground bridge structure did not fall on the critical path for construction of the bridges, negating much of the time saving component.

The viability of this system for application in seismic regions has been confirmed. It is the Authors' opinion that implementation of this, or similar, systems in New Zealand will happen in the near future upon the realisation of the improved seismic performance, reduced construction costs, and the enhanced resilience it offers.

ACKNOWLEDGEMENTS

The Authors would like to acknowledge the contributions of the Mackay's to Peka Peka Alliance team, especially the design and construction teams who were closely involved in investigation and assessment of the Hybrid PRESSS system.

Special thanks must also be given to Dr Alessandro Palermo and his group of students at the University of Canterbury who provided guidance and advice on application of this system which adopts features they are currently researching as part of the Accelerated Bridge Construction and Design research program.

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Appendix 5

Innovative Design and Construction of New Zealand's first 3M Diameter Concrete Bored Piles with Plunged 16T Column Cage in Challenging Seismic, Geotechnical; and Urban Environment

Jamil Khan
Philip Clayton
Matt Zame
Ian Stockdale
Amit Chaudhry
Geoff Brown

THE NEW ZEALAND CONCRETE INDUSTRY
CONFERENCE 2015
Rotorua Convention Centre 8-10 October

INOVATIVE DESIGN AND CONSTRUCTION OF NEW ZEALAND'S FIRST 3M DIAMETER CONCRETE BORED PILES WITH PLUNGED 16T COLUMN CAGE IN CHALLENGING SEISMIC, GEOTECHNICAL AND URBAN ENVIRONMENT

JAMIL KHAN[1], PHILIP CLAYTON[2], MATT ZAME[3], IAN STOCKDALE[4], AMIT CHAUDHRY[5], GEOFF BROWN[6]

[1] Technical Director – Structural Engineering, Beca Ltd.
[2] Technical Director – Geotechnical Engineering, Beca Ltd.
[3] Foundation Manager, M2PP Project, Fletcher Construction Ltd.
[4] Senior Zone Engineer, M2PP Project, Fletcher Construction Ltd.
[5] Project Engineer, M2PP Project, Fletcher Construction Ltd
[6] Technical Director – Bridge Engineering, Beca Ltd.

SUMMARY

Innovation happens when designers and constructors collaborate to put existing concepts together in a new way. By pushing to find innovative solutions in the design and construction of the piles and piers, the M2PP project team achieved significant construction efficiencies and savings as discussed in this paper.

Multi-span bridge piers on the MacKays to Peka Peka (M2PP) Expressway Project are founded on large 3m diameter bored concrete mono-piles. With depths of up to 38m and 60tonne reinforcing cages, these piles are some of the largest bored piles ever constructed in New Zealand.

Designers and constructors collaborated to develop this innovative and challenging solution resulting in robust bridge supports and significant construction efficiencies.

The design approach included:

- Use of hammer-head piers on mono piles, which eliminated the need for pilecaps
- Non-linear pushover analysis modelling of the piles.
- Pile load testing to optimise geotechnical design.
- Restricting the potential plastic hinges to within the piles only.
- Use of bentonite drilling support fluids instead of steel casings.
- Connection detailing for the pier column on oversized pile shafts, using the AASHTO 2013 design provisions.

The construction of the 3m diameter bored piles has given the following construction advantages:

- Nearly 50% reduction in pile numbers resulting in approximately $3 million dollars material savings and about 80 trimmed from the programme.
- Plunging of fully assembled asymmetric column cage into the piles to eliminate a second stage pile-column pour.
- Elimination of permanent steel casings by the use of bentonite support fluids, saving costs, significantly increasing the available skin friction and avoiding lapping of reinforcement in potential plastic hinge zones

- Elimination of pile group /pile caps which minimises lateral spread soil loads on bridge piles

PROJECT BACKGROUND

The MacKays to Peka Peka Expressway (M2PP) is an 18km 4-lane highway that will take State Highway 1 along the Kāpiti Coast. M2PP is the first of the Wellington Northern Corridor RONS projects. M2PP will separate local and state highway traffic and result in safer and shorter trips within and through the Kāpiti Coast - with local and national benefits. It is being built by an alliance made up of the NZ Transport Agency, Beca, Fletcher Construction and Higgins Group supported by Goodmans Contractors, and Boffa Miskell. The project includes 17 bridges comprising multi-span and single span bridges over local roads and streams, including a new 182m long crossing of the Waikanae River.

CHALLENGING ENVIRONMENTS

Ground Conditions

The Expressway alignment traverses through sand dunes and inter-dune peat deposits. The peat is very soft with a high organic content and may be up to 6m thick. The dune deposits are fine, single sized sand, with a high liquefaction potential where saturated. These conditions present the following challenges to the expressway design:

- Peat deposits can cause significant post construction settlements due to high compressibility which, without treatment would have resulted in poor ride ability, settlement of services and adjacent properties, altered surface drainage patterns and increased maintenance.
- The dune and sandy alluvium presents significant challenges due to liquefaction and associated settlement and lateral spreading, particularly to bridge structures.

Seismicity of the Area

The site is located in a highly seismic area and the following active faults are located near the Project.

- The Ohariu fault is between 1 and 3km from the Expressway and has an estimated MCE (maximum considered earthquake) magnitude of M7.2 at a return period of 2000 years.
- The Wairarapa fault is around 30km further from the Expressway but has an estimated MCE magnitude of 8.2 at a return period of 1200 years.

Figure 1: Critical active faults around the site

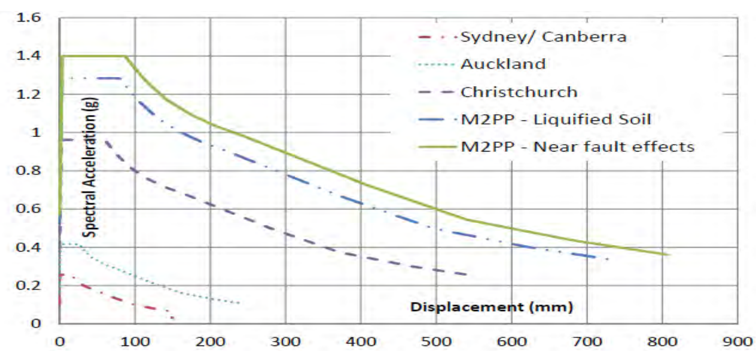

Figure 2: A comparison of Spectral Acceleration of different cities with Project site for 1/2500 years seismic events.

To appreciate the high seismicity of the project site the design spectral acceleration curve is compared against other locations in New Zealand and Australia in Figure 2.

Seismic Design Criteria

The bridges are to be designed to achieve the requirements of the NZTA Bridge Manual 2nd Edition [7]. The bridges are lifelines post-earthquake and any damage shall be repairable within 12 months.

More specifically the bridges are to be designed to:

- Have no damage under SLS1 (1/25 years) seismic events.
- Have some minor damage under the SLS2 (1 /100 years) seismic events. Damage to such components is to be cleared and access restored within 24 hours for full traffic use.
- Have some damage to the structure under Ultimate Limit State design (1/2500 years) earthquake events. The bridge should be useable by emergency traffic within 3 days. Permanent repair to reinstate the design capacity for both vehicle and seismic loading should be feasible and should be economically viable and able to be accomplished within 12 months.
- Prevent total collapse under the maximum credible earthquake (MCE) by ensuring sufficient ductility capacity in the yielding elements or where appropriate providing secondary load paths.

Urban Environment

The Expressway is a new feature in the landscape and by its nature is strongly horizontal. The expression of that horizontality is acknowledged whilst also recognising that it hovers over the ground where it crosses local roads. Where bridges interface with local roads the concept is to translate its supporting armature of columns and beams into a single and fluid shape to simplify the appearance of the structure rather than drawing attention to it.

Therefore, the bridges should be:

- Consistent in their form so they register as a 'family' and provide some visual continuity within the local environment.
- Expressed as simple forms that sit across the changes in landscape and are not seen as strong statements in their own right.
- Consists of elements like piers, cross heads, decks and barriers, which when united create one sculptural form and ensure services are concealed from view.
- Of a form that the underside is visually appealing, to recognise the primacy of the local road user's experience in design considerations.
- Designed in such a way that the intersection of the piers with the ground is in concert with the abutment forms and materials.
- Designed to enhance the quality of the space beneath the bridges, including the use of natural light penetration.

BRIDGE FORM

The multi-span bridge beams are single hollow core or super tee beams and are simply supported and connected via transverse post-tensioning or a concrete topping slab. In the longitudinal direction the bridge deck will slide over the abutments and in the transverse direction the deck is restrained by shear keys under seismic loads.

Piers are single columns with precast crossheads. Each pier is supported by a single bored, cast in-situ concrete mono-pile, which resists both longitudinal and transverse seismic effects. The abutments are cast-in-situ concrete, supported by steel H-piles, MSE walls and ground improvements, which resist only transverse seismic inertia loads from the deck. Figures 3 and 4 show a typical cross-section of a multi-span bridge.

Figure 3 – Cross section of multi-span bridge with Super-T beam

Figure 4 – Cross section of multi-span bridge with SHC beam

Figure 5 – Visual of New 182m long crossing of Waikanae River

NEW ZEALAND'S FIRST 3M DIAMETER BORED PILE

The designers and constructors collaborated to develop this innovative and challenging solution to use 3m diameter bored concrete piles, to provide efficient support to the bridge piers and also give significant efficiencies in construction and design. The ability to construct these piles was a result of:

- Innovative design of hammer-head piers on mono plies.
- Non-linear soil-structure interaction using multi-linear soil springs expressed as P-Y curves.
- Pile load testing and analysis to optimise design parameters.
- Restricting the potential plastic hinges to within the piles.
- Detailing pier-pile connection without pilecap following AASHTO guide lines.
- Pushing NZ bentonite drilling technology to a new level, including fluid trials and new tools.
- Developing understanding and innovative systems in the lifting of 60T reinforcing cages "the step change".
- Development of specialist plunge column cage guide frames and lifting gear.
- Extensive concrete mix trials to ensure workability for long periods to allow plunging.
- Complex large diameter pile cage and column cage fabrication and transport systems.
- Risk management of challenging tremie pour and plunge column operation.
- 2.1dia 40m deep test piles with Osterberg Cells.
- The use of friction pile design principles to provide flexibility in pile design to accommodate changes in structural design loading and the unpredictable presence of silt layers within the founding stratum.

We believe that this is the first use of 3m diameter bored reinforced concrete piles in New Zealand although we are aware of shafts/caissons or non-bored piles of a similar size e.g. Arthurs Pass Viaduct.

DESIGN ASPECTS

<u>Hammerhead Piers on Mono Piles</u>

These bridges sit in an urban environment, where space and aesthetics are very important. The bridges are designed to enhance the quality of the local road space beneath the overpass by maximising natural light penetration. Hammerhead piers have been used because they're aesthetically appealing, occupy less space and provide more light and room for the traffic underneath the bridges.

The bridge piers were originally designed to be supported by a group of piles and a pilecap, buried in the ground. The Alliance team challenged this concept, instead utilising a large mono pile system for each pier. The decision to plunge the pier cage naturally followed.

Preliminary design indicated that we needed 2.7m~3.0m diameter piles to support each pier using a mono pile system. After detailed investigation a 3m diameter pile was adopted to allow the plunging of the architectural shaped pier cage. As both pier core reinforcement and pile reinforcement are in circular shape, the adopted system has flexibility and construction tolerances.

The bridge piers supported on large mono-piles act as cantilever structures supporting static and seismic inertial loads as well as resisting lateral ground movement loads under extreme seismic loads. The absence of a pilecap reduces the lateral loads applied by laterally spreading ground in the liquefaction design case.

<u>Site Specific Seismic Hazard Assessment</u>

A site specific seismic hazard assessment (SSSHA)[9] was undertaken for the Expressway corridor. The assessment utilised the New Zealand National Seismic Model and appropriate attenuation relationships to estimate the earthquake shaking hazard at the site. The SSSHA recommends that the project adopt the recommended local Z-factor of 0.4 (NZS 1170.5[6]) but utilise modified R factors as set out in Table 1. The values in brackets are NZS1170 recommended values for comparison.

The SSSHA also proposed modified response spectra, specific geotechnical design values and specific MCE events for the site. Figure 6 shows a comparison of the MCE response spectra for Ohariu fault (with Median + 1 standard deviation), Wairarapa fault (with Median + 1.5 standard deviation), 1000 and 2500 year return period response spectra obtained in this investigation and the NZS 1170.5[6] response spectra for the same return periods for subsoil Class D.

Table 1: Return Period Factor R

Required Annual Probability of Exceedance	Rs for SLS or Ru for ULS
1/250	0.80 (0.75)
1/500	1.00 (1.00)
1/1000	1.25 (1.30)
1/2500	1.65 (1.80)

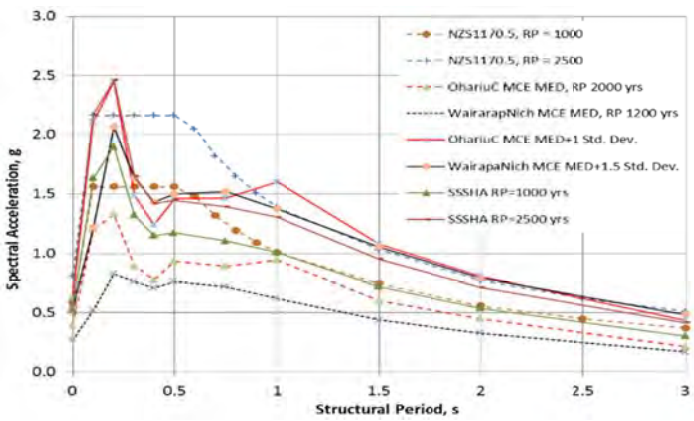

Figure 6: Comparison of Spectra for Site Sub-soil Class D

Soil-structure Interaction

Design of piles for the effect of seismic motions is generally performed by using a soil-pile interaction model in which a vertical element is supported by a series of horizontal linear elastic Winkler spring elements. The element represents the performance of the pile, and soil properties are represented by linear-elastic spring constants. The effect of horizontal seismic motions on piles are allowed for by incorporating a horizontal force at the top of the pile, which is equivalent to the inertia force from the superstructure.

In the past modelling of non-linear soil responses has been undertaken by approximating the non-linear behaviour of soil with linear springs. Multi-linear soil springs are more rarely used due to the limitations of most structural engineering analysis packages. On the M2PP project, the soil response was modelled using p-y-curves derived in the geotechnical software package L-Pile[5]. The soil profile at each pile location was simplified into horizontal layers with similar strength/density and the most appropriate p-y response model selected. Response models were selected as follows:

- Sands and silty sands utilised a model published by Reese, Cox and Koop[13]
- Plastic silts and clays utilised a model published by Matlock[14]
- Liquefied sand utilised a model published by Rollins, Gerber and Ashford[15]

A range of soil properties comprising bulk density (overburden stress), friction angle or undrained shear strength were then utilised to factor the soil model and generate appropriate P-Y curves. For each layer ‘nominal’, ‘hard’ and ‘soft’ values were derived to reflect the likely range of soil properties. This approach was taken to consider a ‘most likely’ behaviour alongside a case likely to result in upper bound deflections i.e. the ‘soft’ case and upper bound actions i.e. the ‘hard’ case. The P-Y curves were then utilised in two different ways, either within subsidiary analyses to derive a pile top response matrix or as inputs to a global analysis.

At the pier locations, where reinforced concrete mono piles were employed, the spring characteristics were derived in L-Pile by modelling the stiffness characteristics of the pile (refer to Figure 7) and applying shear and flexure increments to the pile head.

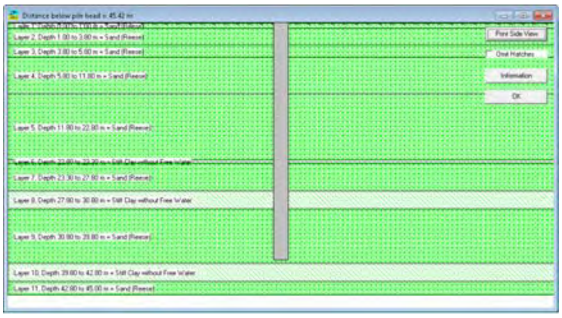

Figure 7: Screenshot of a soil profile [3] for pile modelling

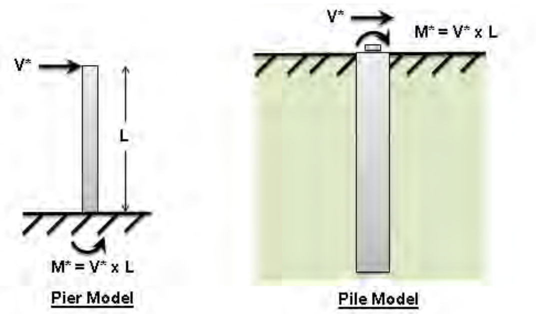

Figure 8: Application of Shear and Flexure to pile in L-PILE[3]

The flexure and shear effects were proportioned to match those expected by the structural system, e.g. as shown in Figure 8 a cantilever moment being proportional to the distance from the pile head to the centroid of the seismic mass and application of the base shear. At each increment shear-displacement and moment-rotation relationships were recorded, until yielding of the pile occurred. The resulting shear-displacement and moment-rotation plots defined the characteristics of the pile head spring.

Pile Axial Capacity

To allow flexibility in pile capacity and to accommodate irregularly occurring silt interbeds within the foundation stratum, the pier piles are designed assuming they derive their capacity in friction. Assuming heavily over-consolidated conditions and a rough interface allowed the adoption of skin friction values in the dense gravels of up to 250kPa. With limited historical precedent for heavily loaded piles in these soils testing was considered prudent, additionally, the increased certainty that results from a load testing program allows the adoption of higher strength reduction factors.

Pile Load Testing

Two, near full scale (2.1 m diameter, 32-35m long) fully instrumented test piles were employed on the project. One installed at the site of the Waikanae River Bridge and another at the Te-Moana Bridge. The test piles were instrumented and tested by Fugro Singapore Pte. Ltd[21]. An assembly consisting of two 510 mm Osterberg cells (O-cells) was installed near the toe of each pile approximately 30 m below ground level; three diameters above the pile toe. The typical instrumentation (refer Figure 9 [21]) included:

1. A pair of automated digital survey levels for measuring top of pile displacement,
2. Seven levels of four vibrating wire strain gauges above the O-cell assembly, attached at 90° spacing within the reinforcing cage,
3. Four tell-tale rods at 90° spacing monitored by displacement transducers (attached to the top of the pile) for measuring shaft compression above the O-cell assembly,
4. Four displacement transducers for measuring expansion of the O-cell assembly,
5. Three levels of four vibrating wire strain gauges below the O-cell assembly, attached at 90° spacing, and
6. Four tell-tale rods at 90° spacing monitored by displacement transducers (attached to the top of the pile) for measuring pile toe displacements.

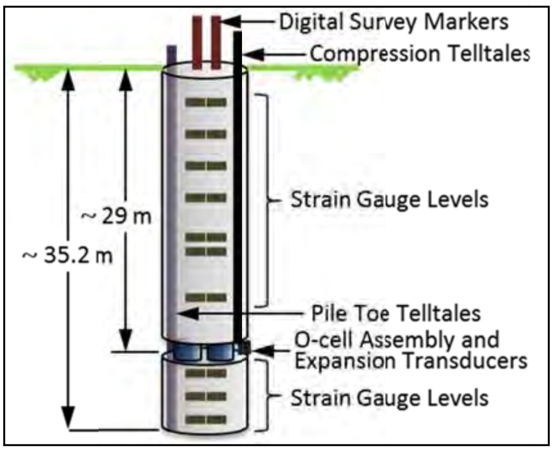

Figure 9. Indicative instrumentation arrangement (not to scale).

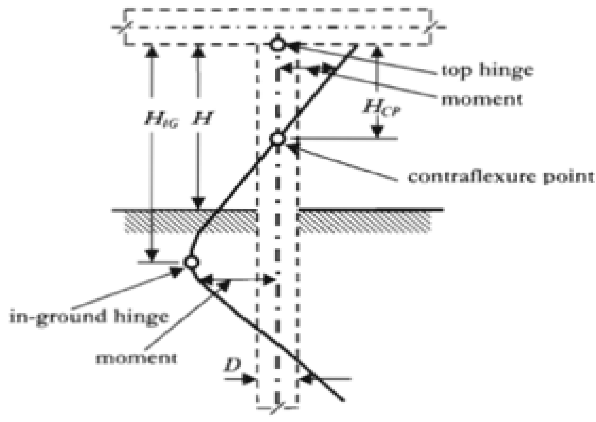

Figure 10: Plastic hinge length for hinge at depth[2]

Potential Plastic Hinges Restricted in Piles

Design economy was achieved by allowing the formation of plastic hinges at depth in the piles. This develops an inelastic displacement capacity in the piles which contributes to satisfying the spectral displacement demands in lieu of an increase in flexural section capacity.

In this regard NZTA Bridge Manual[7,,8] limits the level of plastic hinging by limiting the level of displacement ductility to less than 3 which defines the upper boundary of limited ductility in

NZS3101. The limit on displacement ductility implies that pile hinging is acceptable within the strain limits and resulting curvature limits as defined in NZS3101[4] when assuming limited ductility.

The strain limits associated with the curvature limits appear significantly higher than strain limits of ε_c=0.008 and ε_s=0.015 with ε_c being the limiting concrete strain and ε_s being the limiting steel strain suggested by Priestley, Calvi and Konalsky[2] for piles hinging at depth.

What compensates for the suggested lower strain limits is the difference in the approach of deriving the plastic hinge length required to calculate the inelastic rotation and displacement capacity: Based on NZS3101[4] the plastic hinge length L_p for a pile is hc/2, whereby hc is in this case the pile diameter. Based on Priestley's recommendations [2], L_p can be calculated as:

$$L_p = D + 0.1(H - H_{cp}) \leq 1.6D \quad (1)$$

with H_{cp} = 0 for cantilever columns.

Refer to Figure 10 for definition of terms.

Hence L_p is up to three times greater than the value derived from the code NZS3101.

During displacement-based seismic design the flexural capacity of sections where plastic hinges were intended to form ("ductile sections"), e.g. in the concrete piles, adopted the probable strengths for concrete and reinforcing steel (1.3f'c, 1.1fy). The number and diameter of longitudinal reinforcing bars in the piles and the piers were determined in order to satisfy the seismic demand on the structure across all cases considered. These bars were then detailed, spliced and curtailed in accordance with NZS3101:2006.

Transverse pile and pier reinforcement inside and outside the Potential Plastic Hinge (PPH) regions was designed to NZS3101:2006 for (overstrength) shear, concrete confinement and anti-buckling. The overstrength flexural capacity of the concrete piles and piers for the purposes of determining shear demand was determined in SAP2000 based on the overstrengths for concrete and reinforcing steel, that is, 1.7f'c and 1.3fy. The shear capacity of the piers and pile, on the other hand, used the characteristic strengths for concrete and reinforcing steel, (that is, f_c and f_y) and the design and detailing was in accordance with NZS3101:2006.

The hinges in the piers (which cantilever) always occurred at the base of the pier, while plastic hinges were found to occur between 5m and 12m deep in the piles. The locations of the hinges, and thus the PPH regions in the piles, differed due to different soil profiles and soil conditions (whether the soil was upper bound, lower bound, or lower bound liquefied). As a single pile and pier design for the bridge is to be adopted, a PPH region that envelopes all possible plastic hinge positions, and extended approximately 16.0m in the pile, and 2.1m in the pier, has been adopted. The PPH region does not stray into the pier/pile splice zone.

Pier Supported by Oversized Pile Shaft

Two types of column anchorage failure are possible in an enlarged pile shaft. The bar pull-out of the pile, and column pull-out due to concrete damage caused by a 'plying' action when the confined core of the column rocks back and forth in its shaft. Both can contribute to the anchorage failure of a column. The plying action introduces horizontal forces that can be relatively large near the top and bottom of the anchorage region. These horizontal forces are resisted by the surrounding concrete and the transverse reinforcement in the shaft.

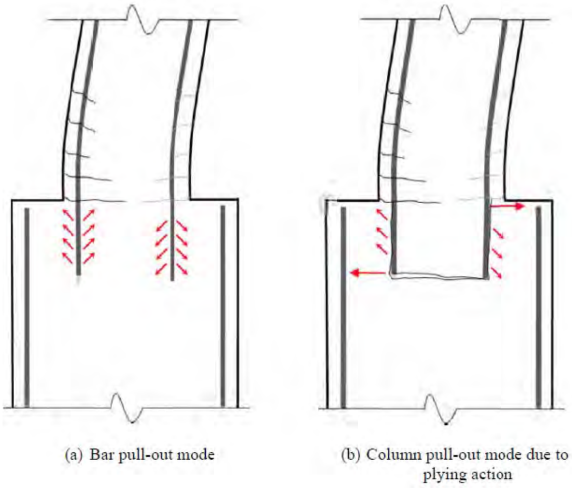

Figure 11: Behaviour of Column Anchorage in Oversized Pile Shaft.

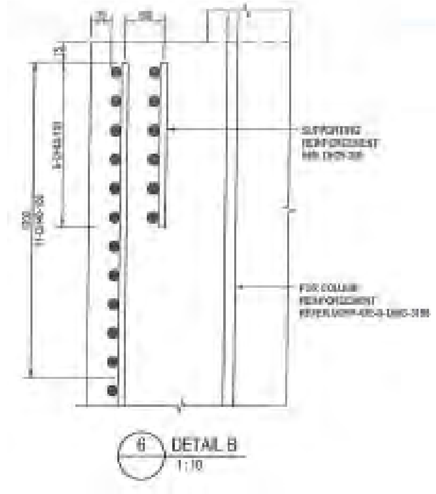

Figure 12: Column – Pile Shaft Connection.

There is no guidance in New Zealand standards to prevent these types of anchorage failures, but this type of pier-pile shaft connection is discussed with details in the draft appendix to the AASHTO bridge design code, 'Precast Bent System for High Seismic Regions- Appendix A: Design Provisions', 2013[22]. This design approach requires additional hoop reinforcement in the top region of the pile.

Use of DH40 Bar Hoops

To force the formation of hinges in the piles at a depth below the pier-pile reinforcement splices zone, the top of pile shaft was designed for high over-strength shear and confinement demands. This led the team to use the 40mm diameter deformed bars as hoops for the piles and 40mm bundle bars as longitudinal reinforcement to meet the reinforcement demand without the excessive congestion. We believe that it is the first use of DH40 bars as hoops in New Zealand.

Tremie Concrete Mix

The concrete for the piles was designed with high flow and longer setting time than normal. This ensured adequate flow around the heavy reinforcement cages and allowed sufficient time to plunge the column reinforcement cage into the fresh concrete. Consideration was also given to the loss of water from the concrete due to highly permeable soil around the piles.

PILE CONSTRUCTION

Drilling Technology

3m Temporary steel casing

3m pile drilling with Liebherr 885 and RT3 Table mount Rig

Concrete pour set up with two pumps

3m pile cage installation

Concrete pour using specially designed hopper

Figure 13; Installation of 3m Diameter Bored Piles

Trials were undertaken to find the best drilling support fluid. This involved trial bores left open for several days with different types of bentonite and/or polymer fluids. For the soil conditions at the Expressway, pure sodium bentonite was the clear winner.

A drilling bucket was specifically designed and built to drill a 3m diameter hole through the underlying local sand, silt and gravels under bentonite. Consideration was made to size and volume of bucket, along with under fluid systems. Some of the “gravels” got up to 600mm diameter.

The mass of the bucket plus the telescoping kelly bar increased the drilling loads up to 28T for a single line pull. This resulted in the use of a Soilmec RT3 setup on 120T Leibherr 885 Crawler Crane. No other crane in New Zealand was suitable for this job as the 885 had 30t line pull on single winch.

Due to the high drilling loads an extensive inspection and maintenance programme was set in place to mitigate risks of high wear on heavily loaded elements including kelly bar, swivels and drive gear.

Cage Fabrication and Lifting

3m diameter pile cages are labour and plant intensive to build, difficult to transport on tight local roads and complex to lift and install. Key features of the cage fabrication and installation included:

- Substantial temporary strengthening of pile cage to avoid cage deformation.
- Extensive welding and testing of each 95kg HD40 hoop and other welds.
- 3D designed and load tested cage clamps for connection of lifting equipment.
- Modified trailers for transportation of over-length and over-wide loads.
- Fabrication of a jig for asymmetrical architectural curved column reinforcing cage.

3m pile cage fabrication

Column cage fabrication in the Jig

3m pile cage delivery

Column cage delivery

Figure 14: Pile and column cage fabrication and delivery to the site

In addition to the above, complex lift planning and temporary works was required to:

- Lift 35T cage sections from horizontal to vertical with two winch lift for 200T crawler crane
- Temporarily support cage sections on the pile casing with clamps
- Connect additional cage sections over pile hole using cherry pickers
- Lower total 60T cage to height hanging off stress bar bridle system
- Withdraw and laydown 16T pile casing with a modified extra wide vibrohammer.

Lift and install 16T pre-assembled column cage to stringent tolerances.

<u>Concrete Testing and Development</u>

A series of concrete tests were conducted to evaluate the duration of the concrete mix workability. Field trials included plunging a reinforcing cage into a 1200Ø concrete pile. This helped to determine what time could reasonably be allowed to plunge the column cage.

Table 2: Cage Plunge Test Results

	Slump at the Plant	Slump measured at site Vs time							
		1145	1245	1345	1445	1545	1645	1745	1845
Slump (mm)	220	200	180	150	125	120	80	80	80
Cage Reached bottom of pile	-	yes	yes	yes	yes	yes	yes	yes	yes

One trial mix used fly ash and the other used a concrete fluidifier. The mix design using fly ash with a number of admixtures was chosen and developed. There was also a concern that the temperature of a straight cement mix in a 3m diameter pile would reach 80˚C over 76hrs, whereas the fly ash mix was estimated to reach 65˚C over 90hrs.

Figure 15: A dummy cage plunging trial in concrete to measure the initial set time of concrete and cage plunging ability with the time

Figure 16: Lifting frame and column cage plunging trial to measure the total time of plunging operation

Plunge Column Cage

Plunging the fully assembled column reinforcement cage into the pile concrete reduced the programme and saved 210 days in preparation and forming column pour connections and 175 days in fixing steel at height across the project. It also made things safer by eliminating confined space work and reducing work at heights. The primary risks associated with this opportunity were:

- Maintaining concrete workability for around 5hours.
- Achieving design tolerances (Horizontal 20mm, Vertical 20 mm).
- High bleed water from a high slump concrete.

Time and motion trials were done on setting up temporary works and plunging the column cage into a dummy hole. Regular reviews of the trials and throughout piling led to refinement and continuous improvement on reducing time and increasing safety and quality assurance.

Top of pile clean up

Column cage plunge frame setup

Column cage lifting and slewing

Column cage introduced into plunge frame

Cage lowered to final position

Plunged column cage after removal of temporary frame

Figure 17: Plunging the fully assembled 16T column cage

<u>Construction Risks and Mitigation</u>

Constructing 3m diameter piles to tight tolerances (Pile = Horizontal 75mm, Vertical 25mm, Verticality 1:75, Column = Horizontal/Vertical 20mm) without any defects has a number of challenges.

After the pile is tremmie poured, it is critical to dismantle the pile access platform, extract the 3m diameter temporary casing, clean away waste slurry and install the guide frame to accept the column cage.

To enable these operations to be efficient and accurate a guide frame was developed. The frame has three parts.

1. A precast concrete base placed around the installed casing and its position surveyed for accuracy.
2. A guide frame on the precast base, which is installed after the casing is withdrawn.
3. A lifting frame connected to the column cage that engages the guide frame ensuring orientation and vertical position.

When the column cage is plunged it is critical that the concrete has good workability. Bleed water from concrete with a high slump is not uncommon and is easily detected by cross-hole sonic log testing. Each pile has four sonic log tubes attached to the reinforcing cage and to date 24 piles have been constructed and all piles have satisfactory sonic log results.

Another key risk included supplying 270m³ of consistent quality concrete within a 2-3hr period. Workshops were held with the concrete supplier and mitigation developed in the form of:

- Three concrete batching plants available in case of breakdown.
- 20 No concrete trucks on turn around.
- Starting concrete pour early in the morning (5am) to avoid traffic delays.
- 2 No boom pumps pumping concrete.
- Every truck is slump tested by a trained concrete technician.
- Controlling the tremie embedment depth and remove pipes regularly to keep fresh concrete on top.

USER'S SATISFACTION

The piles and piers proved to be a success both in their design and construction. The client, New Zealand Transport Agency, is satisfied with the final outcomes for the project. In the following text we include the statements from Principal Project Manager Tony Coulman of New Zealand Transport Agency, Local Member of Parliament, Nathan Guy MP for Otaki and few local community members.

<u>New Zealand Transport Agency</u>

A statement from Tony Coulman, NZ Transport Agency Principal Project Manager

"The collaborative design-construct approach and attention to safety in design by the M2PP Team has delivered exceptional innovation by ensuring that the design was optimised not only for the high seismic forces but for the team's ability to construct these large scale piles and pier columns in a manner that yielded significant savings.

The development of a guide frame and plunging of prefabricated column cages into the bridge foundation piles not only reduced costs and steps in the construction process, but provided significant time savings in delivery, and contributed to improved safety outcomes.

These achievements will contribute greatly towards one of the Transport Agency's core objectives of delivering a cost optimised expressway."[23]

<u>Local Member of Parliament</u>

The Member of Parliament for Otaki, Nathan Guy posted on his website on 25 march 2015

"It's about 18 months since the Kapiti Expressway got underway and the progress how been absolutely amazing. Last week I had the privilege of visiting many of the sites along the route that ended the tour at Ōtaki Pre-Cast Yard. The project is on target to complete the 18km route that includes 18 bridges and end-to-end cycleway, walkway and bridleway by mid-2017.

This year there is a real focus on bridges with the foundation team having so far pioneered some of the country's largest and deepest bored piles (3m diameter 60 tonne reinforcing cages) in difficult local soil conditions.

Soon the country's biggest crosshead beam will be poured at Otaki weighing 170 tonnes and "Super T" beams will be made at the new facility at Otaihanga to support the Waikanae River bridge decks. This will be an exciting development to see them installed and then for all of us to drive on it."[24]

<u>Local Community</u>

Sue Gibbons;	"Would be interested in knowing how the Bentonite works in the piling operation and how it is extracted for re use, thanks".
Merrell Greenwood	"Simply love these newsletters and then I can see more intelligently as I drive around what is happening. I think you are amazing with all the care you are taking and the explanations you give about everything. Thank you so much for taking the time to tell us so much, we truly appreciate it, and read all with interest."
Brian Sherborne	"Awesome effort. As a technology teacher at the Raumati Technology Centre I appreciate the technical aspects. As a resident of Rata Rd and close to the construction area I appreciate being informed about aspects of the project that may impact on us. Well done. "

CONCLUSIONS

Innovation happens when designers and constructors collaborate to put existing concepts together in a new way. By pushing to find innovative solutions in the design and construction of the piles and piers, we had achieved significant construction efficiencies and savings.

Collaboration between designers and constructors enabled the use of 3m diameter piles with plunged columns cages for the multi-span bridge piers.

This approach allowed the M2PP Alliance Team to construct 24 No piles safely and efficiently saving the project over $6,300,000 and 120 days. Innovative design, early planning and thorough trials have facilitated a highly effective construction method and produced high quality piles and pile-column interface connections.

Figure 18: Completed columns for hammered-head piers of Waikanae River Bridge

ACKNOWLEDGEMENTS

The authors would like to acknowledge the numerous and continuing contributions of the NZTA, other Alliance partners and suppliers in the development and implementation of the innovative approach taken on this project.

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