Use of Ductile Iron Pipe Piles for a New Shared Use Path Bridge over Maribyrnong River in West Melbourne

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ABSTRACT

Ductile Iron Pipe Piles (DIPPs) were adopted as the preferred foundation piles for a new Shared Use Path (SUP) Bridge over the Maribyrnong River in West Melbourne. The DIPPs were selected in lieu of the originally proposed CFA piles due to ease of installation and restrictive site constraints.

As part of the foundation design a pile group analysis was undertaken to assess pile actions and pile head deflections. Nominated test piles were tested with high-strain dynamic load testing using Pile Driving Analyser (PDA) with CAPWAP analysis using signal matching. Reported CAPWAP analysis on a representative blow from each test pile showed that the mobilised geotechnical capacity exceeded the required geotechnical strength for both abutment and pier locations.

This paper presents description of the design including pile group analysis undertaken using two methods, description of the DIPPs and construction methodology.

A DIPP was for the first time used on a VicRoads project on the recently completed West Gate Distributor - Stage One Project.

Keywords: Pedestrian Bridge, DIPP, Pile Foundations, Pile Group

1. PROJECT OVERVIEW

A new SUP bridge was constructed over the Maribyrnong River in West Melbourne to improve the amenity for cyclists and pedestrians travelling on one of Melbourne's busiest cycle routes. The bridge was designed by SMEC Australia Pty Ltd (SMEC) in collaboration with Pile Design Solutions Pty Ltd, who had previous experience with the design of DIPPs on building projects. The Project was constructed by Fulton Hogan for the end client, VicRoads. This new 3 span, 190m long and 4.5m wide bridge replaced the existing narrow SUP on the adjacent Shepherd's Bridge, which was widened and strengthened to support increased freight movements to the Port of Melbourne as part of the West Gate Distributor – Stage One Project.

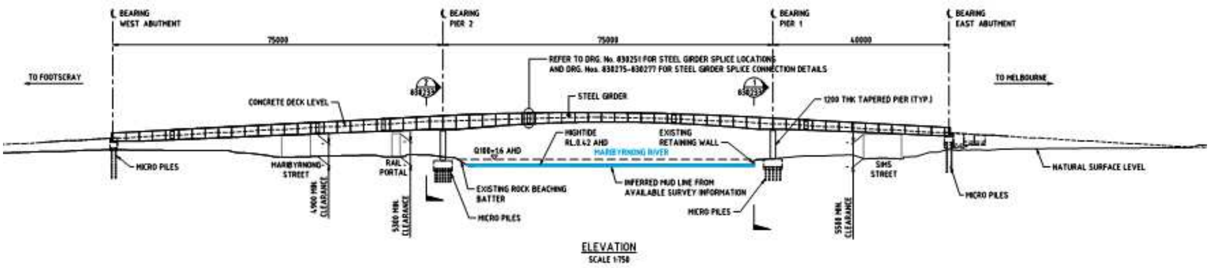

Figure 1 – Elevation of the new SUP Bridge

The SUP bridge comprises two abutments and two piers denoted as East Abutment, East Pier, West Pier and West Abutment. During the tender and preliminary design stages, Continuous Flight Auger

(CFA) Piles were proposed as the foundation system to support the new SUP Bridge. However, during the later stages of detailed design an alternative foundation solution comprising driven 170mm diameter DIPPs was proposed and adopted for supporting the bridge abutments and piers. Given the proximity of the new SUP bridge to the existing Shepherd Bridge and other site constraints including presence of services, DIPPs were the preferred foundation solution compared with CFA and other pile types including precast driven piles. DIPPs were preferred due to the ease of their installation i.e. light weight and more mobile equipment, in addition they were quicker to install and presented cost savings to the project compared to CFA. Furthermore, low vibrations during pile installation and its potential impacts on the existing adjacent services and Shepherd Bridge made DIPPs more suitable for the project.

2. GEOTECHNICAL CONDITIONS

The bridge site is located in the Yarra Delta region of Melbourne and at close proximity to the boundary between the basaltic flows associated with Newer Volcanics and the estuarine sediments.

The ground profile varied significantly between the two abutments and the piers located adjacent to the Maribyrnong River. The general ground profile comprised variable thickness of fill, Coode Island Silt (CIS), Fishermens Bend Silt (FBS) underlain by the Newer Volcanics basalt of varying strength and weathering degrees. The depth to basalt rock head vary from 7m (approx.) at the West Abutment to over 30m in the East Abutment. Basalt rock was considered to be suitable founding strata for the proposed DIPP foundations. A detailed description of the regional geology of the Yarra Delta region including description of the above geological units is provided in the Engineering Geology of Melbourne P.223-259 (W.A. Peck et al, 1992)

3. DUCTILE IRON PIPE PILES (DIPPS)

Ductile Iron Pipe Piles (DIPPs) are circular tube piles made of nodular ductile cast iron. The installed piles comprised 170mm diameter, 5m tube sections with wall thicknesses of 9 mm. The nodular cast iron has a 900 MPa compressive strength and 420 MPa tensile strength, with a yield strength of 320 MPa at 0.2% elastic limit. Due to its high carbon and silicon content as well as the annealing process (oxide layer) the cast iron has a much higher corrosion resistance than steel.

DIPPs are displacement piles installed by ramming the tube sections into ground through soil layers and founded in the hard strata to achieve the required capacity. The sections of the pipes feature a conical spigot at the bottom and a precise-fitting plug-in collar at the top. These sections are jointed to form a continuous pile at any length. At the impact energy driving process the pile sections are forced to plug together producing a rigid stiff connection between the sections. Installation of the piles require a lightweight and mobile excavator mounted with hydraulic hammer, suitable for tight working areas. Like other driven piles, pile load capacity can be approximately proven during pile driving.

The piles were driven to effective refusal in basalt. It was assumed that effective refusal would be achieved when penetration is <60mm in 30 seconds of continuous rapid driving using the HB excavator mounted hydraulic rock breaker. Re-strike of piles was carried out the following day after initial driving to ensure there was no reduction in geotechnical strength from ground relaxation. The recorded penetration at the restrike ranged between 0 to 10mm/sec which was well below the <60mm criteria.

Figure 2 – Piling Rig (Eat Abutment) & Installed Group (East Pier)

DIPPs for the SUP bridge were arranged in the following groups:

- Two groups of 4 piles at the abutments
- Two rows of 5 piles on each side (north and south) of the eastern pier, with an additional two piles installed in the middle to reduce settlement to acceptable limits
- One row of 6 piles on the north side of the western pier and another group of 11 piles in two rows on the south side of the pier, an additional two piles are also installed in the middle as per the eastern pier. One of the central piles was damaged during installation, so an additional pile was installed 300mm away. This pier foundation has an eccentricity as required to avoid clashing with an existing pipeline.

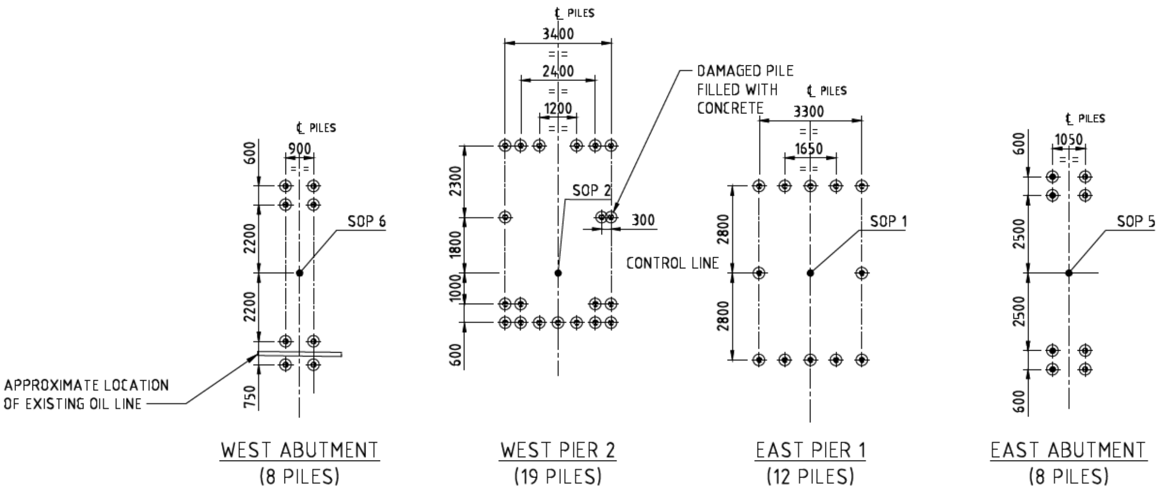

Figure 3 - Foundation layout plan

A corrosion assessment was undertaken to ensure that the piling system complied with the design life requirements of the Specification. Accordingly, a 3mm corrosion allowance was made in the design as per durability requirements.

The piles are a composite system having a steel wall and being filled with 50 MPa super workable concrete. During installation, the first section of the pile has a shoe attached to it to facilitate driving activities. The piles are socketed to enable subsequent lengths to be inserted and driving activities to continue.

To improve the bending capacity of the top section of the DIPPs, 5m long 100 UC sections were inserted into the concrete which is poured into the hollow pile sections to produce a composite pile section. The UC section is designed to be installed with the strong axis parallel to the bridge at the abutments and transverse at the piers to assist in resisting the lateral loads. The UC section has a steel plate welded to the top of the pile and Reid bars screwed into the plate to provide fixity into the pile cap. UC sections were specified to be installed for all piles.

4. PILE GROUP ANLAYSIS

Compared to the original CFA piles, DIPPs are small diameter slender foundation elements. Based on this there was a concern associated with the capacity of the piles to resist lateral loads associated with wind and earthquake cases. It was therefore critical that pile groups were analysed using rigorous methods to obtain a relatively accurate estimate of the pile loads and actions.

During the design stage Piglet software program was used to analyse the pile group behaviour for both piers and abutments. At the abutment locations, piles were also modelled in Plaxis 2D to assess the impact of the abutment construction and abutment backfilling. Post construction of the bridge and in preparation for this paper, further analyses were completed using finite element software package PLAXIS 3D. The further analyses have been completed for pile groups at east pier where the depth to basalt rock exceeded 30m resulting in pile lengths of over 30.

It is worth noting that DIPPs were designed as end bearing piles assuming that for geotechnical capacity the piles would mainly rely on end bearing for the piles founded on basalt rock. The assumption was made on the basis that a gap would likely be formed around the pile due to the enlargement of the pile segments at the connection between each subsequent segments. However, pile load test results achieved significantly higher mobilised shaft resistance indicating that either there was no gap formed around the piles or potentially the enlargement at the connections contributed to increased resistance.

A brief description of the completed analyses together with the results for each analysis are presented in the following sections.

4.1 PIGLET ANALYSIS

PIGLET is a relatively easy to use spreadsheet program developed for analyses of load and deformation response of pile groups under general loading conditions. PIGLET allows one single soil layer to be included in the analysis with soil modelled as linear elastic material. The soil stiffness can be assigned to vary linearly with depth for the material above the pile toe. For the soil layer at the pile toe a different stiffness value can be assigned. PIGLET however, does not have the capability to model soil strength parameters and therefore the analysis does not include the overall stability of the piles and general geotechnical failures.

The following soil and pile properties were adopted in the PIGLET analysis for the east pier piles.

Table 1- Geotechnical parameters adopted in PIGLET Analysis

Soil Parameters				Pile Properties	
Shear Modulus (kPa)	**Shear Modulus Gradient (kPa/m)**	**Shear Modulus Below Base (kPa)**	**Poisson's Ratio**	**Pile Diameter (mm)**	**Yung's Modulus (kPa)**
2,310	500	38,000	0.3	170	$5.1x10^7$

The East Pier included 12 piles arranged in a 5-2-5 layout as shown in Figure 3. During the design stage, a maximum pile length of 27m was estimated based on the available information, however based on the actual ground conditions on site, most piles at the east abutment and east pier were installed to depths of ranging between 29-34m. The pile heads were extended by 300mm into the 1.9m thick pile cap and welded to bolted plates. Based on this pile heads were modelled as fixed in the analyses. In the subsequent analyses undertaken for this paper, as built conditions were modelled and analysed.

The following ULS loads (at the base of the pile cap) were adopted in the pile group analysis. Loads were provided by the project structural engineer.

Table 2– Pile Group Loads (Top of the Pile cap)

Load Case	Vertical Load (kN)	Horizontal Load (kN)	Moment (kN-m)
1	8390	0	0
2	6015	500	4100

Both cases were checked during design stage, with case 1 governing the axial capacity of the piles and case 2 governing the design in terms of lateral loads. In this paper, only results for case 2 loading have been presented for clarity.

Analysis results from PIGLET for east pier are presented in Table 4

4.2 PLAXIS 3D ANLAYSIS

To validate the results of original PIGLET analyses, east pier pile group were modelled in PLAXIS 3D. PLAXIS 3D is a Finite Element Analysis (FEA) package, developed for the analysis of deformation and stability in geotechnical engineering. The piles were modelled using 'embedded beam' in PLAXIS while the pile cap was modelled 'plate' element with concrete properties. Geotechnical parameters adopted in the analysis are presented in Table 3.

Table 3– Assumed Subsurface profile & Geotechnical Parameters

Material/Layer	γ (kN/m3)	Cu (kPa)	C' (kPa)	Φ' (degree)	E ' (kPa)	v'	Ult. Skin Friction (kPa)
Fill	18	75	5	25	25,000	0.35	40
CIS	17	25	2	25	2,500	0.35	-
FBS Upper, stiff silty clay	18	75	5	26	25,000	0.35	40
FBS silty/clayey sand	19	-	3	28	30,000	0.3	50
FBS lower, very stiff silty clay	19	100	10	26	30,000	0.35	50
Weathered Basalt	26	-	50	33	100,000	0.2	300-600

The following construction stages were modelled in PLAXIS 3D;

- Initial – Generate initial stresses
- Install piles
- Excavate to base of pile cap level
- Construct Pile Cap
- Apply bridge load

An overall fine mesh was generated for the for the model with local refinement around the pile cap and along the piles.

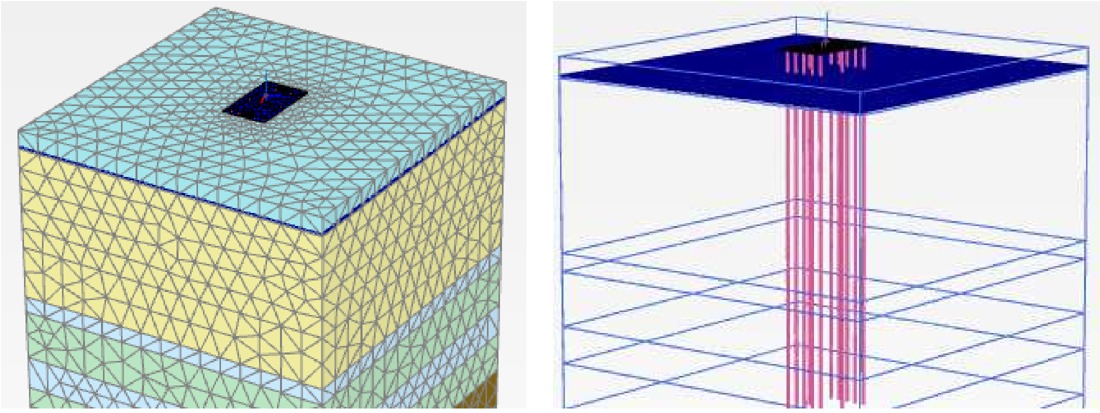

Figure 4 – PLAXIS 3D generated mesh & Pile Group

Results of the PLAXIS 3D analyses indicate generally smaller pile actions and pile head settlement/ deflection compared to PIGLET. A summary of PLAXIS 3D pile group analysis together results from PIGLET analyses is presented in Table 4.

Table 4– Summary of Pile Group Analysis from PIGLET and Plaxis 3D

Pile No	Axial Force (kN/pile)		Max Bending Moment (kN-m)		Max Vertical Deflection (mm)	
	PIGLET	PLAXIS 3D	PIGLET	PLAXIS 3D	PIGLET	PLAXIS 3D
1	385	347	32	31	13	11
2	312	340	29	25	13	11
3	293	338	28	27	13	11
4	312	341	29	27	13	11
5	385	351	33	30	13	11
6	734	668	33	35	18	25
7	614	665	29	30	18	25
8	583	655	28	31	18	25
9	614	655	29	32	18	25
10	734	668	33	33	18	25
11	524	520	31	34	15	18
12	524	520	31	32	15	18

There was a reasonably good match between the calculated pile actions and pile head settlements between PIGLET and PLAXIS 3D analyses.
From the PLAXIS analysis it was observed that the mobilised toe resistance from PLAXIS analysis were in the range 10-70kN with virtually no pile toe movement indicating that pile head settlements were predominantly due to contribution from the elastic shortening of the piles.
It is noted that the calculated settlements are based on ultimate limit state loads for consistency and comparison purposes. During the design stage the pile head settlements were also checked under serviceability loads.

5. PILE LOAD TESTING

Nominated six piles (one at each abutment and 2 at each pier) were tested using high-strain dynamic load testing with Pile Driving Analyser (PDA) and CAPWAP analysis using signal matching. The force and velocity data obtained from the PDA test are used in CAPAWAP analyses to estimate the end bearing and shaft resistance of the pile as well as the distribution of the shaft resistance and pile head displacement. Reported CAPWAP analysis on a representative blow from each test pile

showed that the mobilised geotechnical capacity exceeded the required geotechnical strength for both abutment and pier locations.
Pile test results are summarised and presented in Table 5.

Table 5– Pile Test Results

Test Pile ID	Pile Length (m)	Ultimate Geotechnical Strength (kN)	Test Load from PDA (kN)	Test Load from CAPWAP (kN)	Total Shaft Resistance (kN)	Mobilised Toe Resistance (kN)	Pile Head Settlement at design Load (mm)
2EA	29.5	430	1242	1224	1183	41	3
1EP	29.9	1340	1703	1850	1698	152	13
2EP	29.8	1340	1857	2076	1850	226	13
7WP	11.5	1380	2063	2180	2010	170	7
8WP	7.6	1380	1538	1568	1286	282	8
3WA	11.3	740	1528	1584	803	780	4

Pile test results indicate significantly large shaft resistance of 300-600kPa (near the pile toe) for the west abutment piles. It is noted that thick layers of XW-HW basalt was encountered at approximate depth of 7m bgl at the west abutment. The high shaft resistance from the test results may be attributed to the piles having penetrated through to the XW-HW at the west abutment. For the overlying soils of FBS unit, a shaft resistance in the range 50-75kPa are generally considered reasonable.

Review of PLAXIS 3D analysis results compared with pile test results for east pier indicate a reasonably good match between the pile test results and the calculated mobilised skin friction, end bearing and pile toe settlement.

6. EMBANKMENT LOADING

At the abutment locations, there was a concern relating to additional lateral loading due to placement of fill behind the abutment headwalls. To assess the impact of the fill placement on to the abutment and abutment piles, the construction staging was modelled in PLAXIS 2D with the fill placed in stages behind the abutment. Impact of the abutment fill on to the abutment was found insignificant.

7. SUMMARY & CONCLUSION

Ductile Iron Pipe Piles (DIPPs) were selected as an alternative to the originally proposed CFA piles for the new SUP Bridge over the Maribyrnong River in West Melbourne. Pile group analysis was undertaken using PIGLET and PLAXIS 3D. Generally, there was good match between the two analyses methods in terms of pile actions pile head settlements. Nominated test piles were tested with high-strain dynamic load testing using Pile Driving Analyser (PDA) and CAPWAP analysis using signal matching. Reported PDA and CAPWAP analysis on a representative blow from each test pile showed that the mobilised geotechnical capacity exceeded the required geotechnical strength for both abutment and pier locations. Pile head settlements for tested piles at the design load were in the similar range as the calculated settlements PIGLET and PLAXIS 3D, but generally smaller. Pile head settlement at serviceability load was well below the accepted criteria. The piles were designed to rely mainly on end bearing and conservatively ignoring the contribution from the shaft resistance. This assumption was made on the basis that due to the enlargement of the pile segments at the connections, there would be a gap formed around the pile between the connections. However, the test results indicated that piles achieved significantly high shaft resistance and pile toe capacity was not mobilised in full. On average 90% of the pile geotechnical capacity could be attributed to the shaft resistance based on the test results.

8. AKNOWLEDGEMENTS

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9. REFERENCES

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