Kumpulan IKRAM Sdn Bhd

Blok 5, Unipark Suria,
(Formerly known as Taman Ilmu Ikram)
Jalan IKRAM-UNITEN,
43000 Kajang, Selangor Darul Ehsan
Tel: (603) 87383291, Fax: (603) 87365406

FINAL REPORT

VOLUME I : EXECUTIVE SUMMARY REPORT

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1.0 Introduction

1.1 General

1.1.1 Kumpulan Ikram Sdn Bhd (hereinafter referred to as “IKRAM”) was commissioned by Jabatan Kerja Raya (hereinafter referred to as “JKR”) to carry out a study on “An Independent Design Check of The Pier at Viaduct on Federal Route FT180/001/40 West Port – North Port, Selangor Darul Ehsan”.

1.1.2 This study involved mainly design review check, crack monitoring and material testing on the eight (8) numbers of the affected piers which were reported to have cracks by JKR. The study commenced on 4th of April 2011 and was completed on 4th of August 2011. The Terms of Reference (TOR) by JKR is attached in Appendix A.

1.1.3 This Final Report describes and discusses the works carried out by IKRAM during the four (4) months of the study. This Final Report consists of three (3) volumes of documents:

i.	Volume I	:	Executive Summary Report
ii.	Volume II	:	Design Review
iii.	Volume III	:	Crack Mapping and Material Test Report

1.1.4 Volume I is the summary of the study carried out by IKRAM and it focused on discussing the findings and the subsequent recommendations. Volume II is the design review report which contains discussions on the bridge design and includes the calculations and analysis of the affected piers.

1.1 Objectives

1.2.1 The objectives of this study, as stated in the TOR, are as outlined below:-

i. To do detailed crack mappings and material tests on the affected piers.

ii. To recommend immediate short-term measures to ensure the safety of the affected pier structures, if necessary;

iii. To propose long-term remedial works to the affected pier structures, if necessary;

iv. To recommend any further study works that need to be carried out for the remaining structures.

1.2 Scope of Works

1.3.1 The scope of works required to achieve the aforementioned objectives are in accordance with the Term of Reference (TOR) given by JKR and are as outlined below:-

i. To carry out independent structure design check on the affected pier structures which were completed in 1999 in accordance with the design version of bridge design codes i.e. BS 5400 and BD 37/88;

ii. To carry out detailed condition surveys, crack mappings and material testing on the affected piers;

iii. To propose rehabilitation or strengthening work design for the affected piers, if necessary. The work proposal shall include tender drawings, specifications, bill of quantities and the aforementioned shall be endorsed by a Professional Engineer.

2.0 Independent Design Review

2.1 This task presents the assumptions, methodology and results of the independent design review. The desk study is based on the as-built drawings, design basis and information made available at the time for this design review.

2.2 According to Table 1, three (3) piers, namely P-11A (Type P1-C), P-25 (Type P1-A) and P-33 (Type P1-A) have been selected for this design review after examining their span configuration and structural form. :

Table 1: Details of the Cracked Pier

PIER		Pier Type	Pier dia. (m)	Cross head Depth (m)	Piling	SPAN LENGTH / PRECAST BEAM TYPE	
ID	**TYPE**					**LHS (m)**	**RHS (m)**
P-10A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^{n}-UM + 6^{n}-M	28.05m / 2^{n}-UM + 6^{n}-M
P-11A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^{n}-UM + 6^{n}-M	28.05m / 2^{n}-UM + 6^{n}-M
P-12A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^{n}-UM + 6^{n}-M	28.05m / 2^{n}-UM + 6^{n}-M
P-13B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.17m / 2^{n}-UM + 6^{n}-M	21.85m / 2^{n}-UM + 6^{n}-M
P-14B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.85m / 2^{n}-UM + 6^{n}-M	21.83m / 2^{n}-UM + 6^{n}-M
P-15B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.83m / 2^{n}-UM + 6^{n}-M	21.78m / 2^{n}-UM + 6^{n}-M
P-25	"T"	P1-A	3.0	3.5	6-Ø1200	35.05m / 12^{n}-U	28.05m / 2^{n}-UM + 14^{n}-M
P-33	"T"	P1-A	3.0	3.5	6-Ø1200	28.05m / 2^{n}-UM + 14^{n}-M	28.05m / 2^{n}-UM + 14^{n}-M

2.3 Three (3) independent analytical models were established based on BD 37/88, JKR SV20 and JKR MTAL traffic live load criterias to determine the maximum load effects. Three analysis/design approaches have been performed to study the load effect behaviours, namely conventional beam design approach, Strut-Tie Model (STM) and Finite Element Method (FEM).

2.4 The affected pier structures are checked for both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) conditions.

2.5 Based on the as-built drawings and design criteria made available for this review, the independent design review of this study is presented in report **Volume II: Report on Design Review.**

3.0 Crack Mapping

3.1 The scope of work for crack mapping as specified in Terms of Reference (TOR) is to carry out detailed condition survey and crack mappings on the eight (8) number of the affected piers including measuring the crack width and crack length of distresses observed. The pier(s) involved are as listed below:

i. Pier 10 A
ii. Pier 11 A
iii. Pier 12 A
iv. Pier 13 B
v. Pier 14 B
vi. Pier 15 B
vii. Pier 25
viii. Pier 33

3.2 The detailed condition survey and crack mapping commenced on 6th May 2011 and was completed on 8th May 2011. The dimensional survey had been carried out at 6 nos. of affected L-shape pier and 2 nos. at affected of T-shape pier.

3.3 Results from the crack mapping provided detailed information on the extent of distresses as well as the quantity required for the rehabilitation and remedial works. The detailed crack mapping of eight (8) nos. of the affected piers are presented in their standard formats in report **Volume III : Report on Crack Mapping and Material Testing**

4.0 Material Testing

Nominal in-situ material testing and sampling were carried out at selected locations to serve as a random check on the quality, strength and durability characteristic of the existing reinforced concrete elements. The field testing and sampling works commenced on 30th May 2011 and completed on 13th June 2011. All concrete core samples obtained from the site were sent to laboratory for further diagnosis and testing. The laboratory testing was completed on 1st July 2011. The test locations and detailed test results are presented in **Volume III: Report on Crack Mapping and Material Testing**. The field testing and sampling works conducted for the investigation are:

(a) Concrete cover measurement using covermeter at selected reinforced concrete elements to check the concrete cover provision.

(b) Concrete core coring (38 numbers) to extract concrete core samples for crack depth, laboratory concrete core compressive strength, density test, and petrography examination.

   i. Concrete core coring (20 numbers) to extract concrete core samples to determine depth of crack.

   ii. Laboratory testing on concrete cores collected from site (16 numbers) to obtain the concrete compressive strength and density.

iii. Petrography examination on concrete cores collected from site (2numbers) to obtain the ratio of cement content in the cores, to check delayed ettringite deformation (DEF), Alkaline Silica Reaction (ASR), Sulphate Attack, and Carbonation in concrete content of the cores.

4.1 Concrete Cover Measurement (MS 26: Part 3: 1992)

4.1.1 Electromagnetic device, more commonly referred as covermeter, is used for the non-destructive check of steel reinforcement bars location in reinforced concrete elements and to estimate the concrete cover to the steel reinforcement bars. Adequate concrete cover is necessary to ensure the steel reinforcement bars are adequately protected against unfavourable environment attack.

4.1.2 Adequate concrete cover is essential to ensure that the reinforcement bars are protected against corrosion. The depth of concrete cover is greatly dependent on the type of exposure environment. As per the Table 3.3 of BS 8110: Part 1: 1997, the exposure condition of this building is classified as “mild” exposure environment (i.e. concrete surface protected against weather or aggressive water). To meet the durability requirements for “mild” exposure condition, the nominal cover to reinforcement bars should be greater than 25 mm. As stipulated in Clause 3.3.1.1 of the standard, the actual cover to all reinforcement should never be less than the nominal cover minus 5mm (i.e. 25-5 = 20mm).

4.1.3 A total of eight (8) numbers of piers were scanned using electromagnetic cover meter. The minimum concrete cover at each location tested ranged from 19mm to 63mm. The summary of concrete cover test results is shown in **Table 2** below.

Table 2. Summary of Concrete Cover Measurement

No	Test Reference	Pier No	Element	Measured Concrete Cover (mm)
1	CC A01	P 1-10A	Cross Head	37
2	CC A02	P 1-10A	Cross Head	39
3	CC A03	P 1-11A	Column	51
4	CC A04	P 1-12A	Cross Head	35
5	CC A05	P 1-12A	Column	59
6	CC A06	P 1-13B	Cross Head	24
7	CC A07	P 1-13B	Column	57
8	CC A08	P 1-14B	Column	48
9	CC A09	P 1-14B	Cross Head	55
10	CC A10	P 1-15B	Cross Head	37
11	CC A11	P 1-15B	Column	54
12	CC A12	P 1-33	Cross Head	38
13	CC A13	P 1-33	Cross Head	37
14	CC A14	P 1-33	Cross Head	30
15	CC A15	P 1-33	Column	39
16	CC A16	P 1-25	Cross Head	35
17	CC A17	P 1-25	Column	34
18	CC A18	P 1-25	Cross Head	34
19	CC B01	P 1-10A	Cross Head	23
20	CC B02	P 1-10A	Column	58
21	CC B03	P 1-10A	Cross Head	48
22	CC B04	P 1-11A	Column	51
23	CC B05	P 1-11A	Cross Head	36
24	CC B06	P 1-12A	Column	48
25	CC B07	P 1-12A	Cross Head	33
26	CC B08	P 1-12A	Cross Head	39
27	CC B09	P 1-13B	Column	48
28	CC B10	P 1-13B	Cross Head	21
29	CC B11	P 1-14B	Column	46
30	CC B12	P 1-14B	Column	46

Table 2. Summary of Concrete Cover Measurement (cont'd)

No	Test Reference	Pier No	Element	Measured Concrete Cover (mm)
31	CC B13	P 1-15B	Column	48
32	CC B14	P 1-33	Cross Head	24
33	CC B15	P 1-33	Cross Head	27
34	CC B16	P 1-33	Column	37
35	CC B17	P 1-25	Cross Head	42
36	CC B18	P 1-25	Cross Head	42
37	CC B19	P 1-25	Cross Head	37
38	CC B20	P 1-25	Column	43
39	CM 01	P 1-10A	Cross Head	30
40	CM 02	P 1-10A	Column	21
41	CM 03	P 1-10A	Cross Head	23
42	CM 04	P 1-10A	Cross Head	32
43	CM 05	P 1-10A	Cross Head	43
44	CM 06	P 1-10A	Cross Head	43
45	CM 07	P 1-11A	Cross Head	32
46	CM 08	P 1-11A	Column	41
47	CM 09	P 1-11A	Cross Head	31
48	CM 10	P 1-11A	Cross Head	38
49	CM 11	P 1-11A	Cross Head	34
50	CM 12	P 1-11A	Cross Head	31
51	CM 13	P 1-12A	Cross Head	33
52	CM 14	P 1-12A	Cross Head	46
53	CM 15	P 1-12A	Column	34
54	CM 16	P 1-12A	Cross Head	39
55	CM 17	P 1-12A	Cross Head	40
56	CM 18	P 1-12A	Cross Head	41
57	CM 19	P 1-13B	Cross Head	20
58	CM 20	P 1-13B	Column	50
59	CM 21	P 1-13B	Cross Head	40
60	CM 22	P 1-13B	Cross Head	40

Table 2. Summary of Concrete Cover Measurement (cont'd)

No	Test Reference	Pier No	Element	Measured Concrete Cover (mm)
61	CM 23	P 1-13B	Cross Head	45
62	CM 24	P 1-13B	Cross Head	29
63	CM 25	P 1-14B	Cross Head	25
64	CM 26	P 1-14B	Cross Head	47
65	CM 27	P 1-14B	Cross Head	27
66	CM 28	P 1-14B	Cross Head	31
67	CM 29	P 1-14B	Cross Head	50
68	CM 30	P 1-14B	Column	36
69	CM 31	P 1-15B	Cross Head	36
70	CM 32	P 1-15B	Cross Head	38
71	CM 33	P 1-15B	Column	41
72	CM 34	P 1-15B	Cross Head	62
73	CM 35	P 1-15B	Cross Head	46
74	CM 36	P 1-15B	Cross Head	24
75	CM 37	P 1-33	Cross Head	34
76	CM 38	P 1-33	Cross Head	24
77	CM 39	P 1-33	Cross Head	33
78	CM 40	P 1-33	Column	34
79	CM 41	P 1-33	Cross Head	24
80	CM 42	P 1-33	Cross Head	29
81	CM 43	P 1-33	Cross Head	21
82	CM 44	P 1-33	Cross Head	37
83	CM 45	P 1-33	Cross Head	33
84	CM 46	P 1-33	Cross Head	37
85	CM 47	P 1-25	Cross Head	26
86	CM 48	P 1-25	Cross Head	37
87	CM 49	P 1-25	Cross Head	26
88	CM 50	P 1-25	Column	29
89	CM 51	P 1-25	Cross Head	41
90	CM 52	P 1-25	Cross Head	21

Table 2. Summary of Concrete Cover Measurement (cont'd)

No	Test Reference	Pier No	Element	Measured Concrete Cover (mm)
91	CM 53	P 1-25	Cross Head	31
92	CM 54	P 1-25	Cross Head	30
93	CM 55	P 1-25	Cross Head	19
94	CM 56	P 1-25	Cross Head	40
95	CM 57	P 1-33	Cross Head	63

4.2 Concrete Core Sampling and Lab Testing (MS 26: Part 2: 1991)

The semi-destructive in-situ concrete cores sampling were carried out by extracting concrete cores from selected concrete elements with a diamond core bit. Cores collected were sent for laboratory tests to provide the actual structural properties of the materials used at site, such as in-situ concrete strength, degree of compaction, etc.

A total of sixteen (16) numbers of concrete core samples were extracted from selected cross head and column of the pier. The core samples were tested for the concrete compressive strength and density in the laboratory.

4.2.1 Concrete Core Compressive Strength (MS 26: Part 2: 1991: Section 7)

The concrete compressive strength is one of the important properties of concrete, which determines the load carrying capacity of a structural component. The cores were soaked in water, capped and tested in compressive machine in a moist condition to obtain the maximum failure load for the calculation of estimated in-situ cube strength.

The in-situ core strength (or actual strength) cannot be compared directly to the cube strength that would have been tested at the time of placing during construction. The in-situ concrete is likely to have a higher void content and is unlikely to have the same curing that a standard cube should have.

Besides that, the in-situ concrete strength also depends on where the core sample is taken. For example, higher strength can be found near to the base of a column, whereas lower strength is likely to be found near to the top, owing to settlement effects.

Therefore, the BS EN 13791: 2007, "Assessment of In-Situ Compressive Strength in Structures and Precast Concrete Components" allowed the in-situ compressive core strength (or estimated in-situ cube strength) to be lower than the designed characteristic compressive cube strength.

All sixteen (16) number of cores were subjected to the compressive strength test. **Table 4** summarizes the concrete core compressive strength test results. The test results show that the estimated in-situ concrete cube strength is inconsistent and ranged between **26.0N/mm^2 to 52.5N/mm^2** with **mean value of 42.0/mm^2** and **standard deviation of 7.8.**

4.2.2 Concrete Density (MS 26: Part 2: 1991: Section 1)

Concrete density was obtained by measuring the concrete mass over the concrete volume. A good concrete mix proportion and adequate concrete compaction during construction will ensure dense concrete to meet the strength requirement as well as to provide durability protection. In general, reasonably well-compacted concrete should have a density of not less than 2,200kg/m^3.

All sixteen (16) numbers of concrete cores were subjected to density measurement. The density measured ranged from 2,228kg/m^3 to 2,354kg/m^3 which showed every sample's density tested are above reasonably well-compacted concrete density. This reflected satisfactory concrete compaction of these elements during the construction. The summary of density test result is shown in **Table 3** below.

Table 3. Summary of Concrete Compressive Strength and Density Test

No	Test Ref	Pier No	Element	EICS (N/mm^2)	Density (kg/m^3)
1	CC A02	P 1-10A	Cross Head	36.0	2276
2	CC A03	P 1-11A	Column	45.5	2303
3	CC A04	P 1-12A	Cross Head	52.5	2304
4	CC A05	P 1-12A	Column	44.0	2296
5	CC A06	P 1-13B	Cross Head	26.0	2319
6	CC A07	P 1-13B	Column	36.0	2314
7	CC A08	P 1-14B	Column	27.0	2316
8	CC A09	P 1-14B	Cross Head	51.0	2344
9	CC A10	P 1-15B	Cross Head	52.0	2278
10	CC A11	P 1-15B	Column	42.5	2267
11	CC A12	P 1-25	Cross Head	47.5	2304
12	CC A13	P 1-25	Cross Head	42.5	2300
13	CC A15	P 1-25	Column	37.0	2228
14	CC A16	P 1-33	Cross Head	45.5	2336
15	CC A17	P 1-33	Column	48.5	2319
16	CC A18	P 1-33	Cross Head	38.5	2354

Note: EICS = Estimated In-Situ Cube Strength

4.3 Petrographic Examination (ASTM C856: 2004)

4.3.1 The Petrography examination was carried out in accordance with ASTM C856-88 “Standard Practice for Petrography Examination of Hardened Concrete”. The primary objective of petrographic examination is to assess micro-pore structure and void in core sample under a high quality multi-functional microscope (by employing various magnifications up to x300). Petrographic examination also looks for evidence of cracking and other relevant microscopical features in the core sample, such as colour, cement-aggregate bonding, etc.

4.3.2 A total of two (2) numbers of concrete core samples were extracted from selected elements. From the examination results, there was no gel (reaction product) observed on the aggregates and cement matrix, hence, there was no likelihood of any major alkali-silica reaction (ASR) or alkali-carbonate reaction (ACR) occurred in both core samples that is A01 and A14 at this point. There was also no ettringite crystal observed, hence there was unlikeliest of any delayed ettringite formation (DEF) in the both core samples at this point. The estimated water content ratio is around 0.45-0.55 for sample CC A01 and 0.50-0.55 for sample CC A14.

4.3.3 The coarse aggregate is predominantly composed of granite, with a maximum particle size of about 20mm, sub-angular for both sample with wholly hard, white/grey/black, mottled, slightly altered Granite. The fine aggregates is predominantly composed of quartz with maximum nominal size is about 4 mm; sub-angular to angular, size distributions relatively good with hard/dense, white/grey quartz. The cement aggregate bond for both samples is generally good with low capillary porosity between the aggregate interface and cement paste. No crack was observed along the length of the cores for CC A01 and CC A1. As for the cement matrix, the result shows ordinary Portland cement is used and was observed to be well presence of hydration zone of cement grain (Alites).

4.4 Concrete Cores for Crack Depth

4.4.1 The semi-destructive in-situ concrete cores sampling were carried out by extracting concrete cores from selected concrete elements with a diamond core bit. A total of twenty (20) numbers of concrete core samples were extracted from selected cross head and column of the pier. The core samples were measured to determine the depth of the crack observed at the cores.

Table 4 (a) Summary of Core Crack Depth at Column

No	Test Ref	Pier No	Element	Core Length (mm)	Average Crack Depth (mm)	Minimum Crack Depth (mm)	Maximum Crack Depth (mm)
1	CC B02	P 1-10A	Column	155	25	25	110
2	CC B04	P 1-11A	Column	90	45		
3	CC B06	P 1-12A	Column	100	100		
4	CC B09	P 1-13B	Column	110	35		
5	CC B11	P 1-14B	Column	135	45		
6	CC B12	P 1-14B	Column	100	100		
7	CC B13	P 1-15B	Column	110	110		
8	CC B16	P 1-25	Column	130	60		
9	CC B20	P 1-33	Column	120	60		

Table 4 (b) Summary of Core Crack Depth at Pier

No	Test Ref	Pier No	Element	Core Length (mm)	Average Crack Depth (mm)	Minimum Crack Depth (mm)	Maximum Crack Depth (mm)
1	CC B01	P 1-10A	Cross Head	140	30	15	160
2	CC B03	P 1-10A	Cross Head	155	55		
3	CC B05	P 1-11A	Cross Head	80	15		
4	CC B07	P 1-12A	Cross Head	160	160		
5	CC B08	P 1-12A	Cross Head	140	140		
6	CC B10	P 1-13B	Cross Head	80	75		
7	CC B14	P 1-25	Cross Head	140	40		
8	CC B15	P 1-25	Cross Head	130	42.5		
9	CC B17	P 1-33	Cross Head	150	100		
10	CC B18	P 1-33	Cross Head	95	95		
11	CC B19	P 1-33	Cross Head	160	50		

4.5 Ground Penetration Radar (GPR)

Ground Penetrating Radar (GPR) uses high frequency pulse electromagnetic waves to map subsurface information. GPR uses transmitting and receiving antenna, which are dragged along the ground or concrete surface. GPR data with other surface geophysical methods reduces uncertainty in site characterisation. GPR provides the highest lateral and vertical resolution of any surface geophysical method.

The transmitting antenna radiates short pulses of high-frequency radio waves into the ground. The wave spreads out and travels downward. If it hits a buried object or a boundary with different electrical properties, the receiving antenna records variations in the reflected return signal. The principals involved are similar to reflection seismology, except that the electromagnetic energy is used instead of acoustic energy, and the resulting image is relatively easy to interpret. Integration of GPR data with other surface geophysical methods reduces uncertainty in site characterisation. GPR provides the highest lateral and vertical resolution of any surface geophysical method.

Review of the reflected waveform and patterns generated during a scan, form the basis of interpretation. When the antenna passed over a localised feature, the return time for the reflected signal is smallest when they are directly above the feature- corresponding to a peak on the trace. The Ground Penetrating Radar (GPR) technique provides cross sectional images of reflectors associated with sharp changes.

This method involves transmitting and receiving electromagnetic (EM) waves. An EM pulse is sent into the surface, which travels at a speed dependent on the electrical properties of the material through which it passes. Radar waves are partially reflected (and partially transmitted) at interfaces where there is a contrast in dielectric properties. The amount of energy reflected (i.e. the strength or amplitude of the return signal) is dependent on the magnitude of the contrast. The phase of the reflection (negative or positive) is an indication of whether the radar wave is passing from a less conductive layer to a more conductive layer, or vice-versa. This can be very useful in determining which reflector relates to which layer boundary. A conductive response will be gained from steel reinforcing whilst a resistive response will be gained from an air void.

4.5.1 Limitations and Concern

Depth of penetration is reduced in moist and/or clayey and soils with high electrical conductivity. Penetration in clays and in materials having high moisture is sometimes less than 1 meter. Penetration of depth is further reduced in Marine Clay as it will absorb the radar signals. Radar waves cannot penetrate very far through conductors such as salt water.

The GPR method is sensitive to noise like interference caused by various geological and cultural factors. For example, tree roots and other phenomena can cause unwanted reflections or scattering. Cultural sources of noise can include reflections from nearby vehicles and construction activities. Electromagnetic transmission from cellular telephones, two-way radios, television, and radio and microwave transmitter may also cause noise on GPR records.

Penetration of depth reduces in heavy steel reinforcement bar in the reinforced concrete members. Amount of reinforcing steel may make the inspection of lower layers of concrete very difficult.

All data was processed using RADAN release 6 (June 2004). The scanned sections of all lines were filtered using horizontal filters to remove flat lying noise and with band pass filters to remove extraneous radio and atmospheric noise. These enhance the visibility of possible feature of interest and reduce the effects of outside interference. The summary of the ground penetration radar is shown in the **Table 5** below.

Table 5. Summary of Ground Penetration Radar (GPR)

S/No	1	2
Pier Reference	33	25
Reinforcement Size (mm)	32	32
Concrete Cover (mm)	35 to 59	36 to 60
Spacing (mm)	180-240	200-260
Approximate Lap Length- cross head (mm)	1250-1500	1300-1500
Approximate Lap Length- column (mm)	1150-1360	1200-1300
Approximate Lap Length- soffit (mm)	1250-1550	1250-1550

5.0 Crack Monitoring

5.1 Crack monitoring works were conducted to help to identify possible cause(s) of the structural distresses and to establish the scale of distresses. The monitoring readings were taken fortnightly for a period of three (3) weeks. The details of monitoring results are attached in the **Volume III : Crack Mapping and Material Testing**.

5.2 Thirty (30) numbers of demec gauges were installed at selected locations on 28th April 2011. There are twelve (12) sets of crack monitoring readings taken weekly on 5th May 2011, 12th May 2011, 19th May 2011, 26th May 2011, 2nd June 2011, 9th May 2011, 16th May 2011, 23rd May 2011, 30th May 201, 7th July 2011, 14th July 2011, and 21st July 2011. The summary of the crack monitoring results is shown in the **Table 6** below.

5.3 Figure 1 to Figure 7 show the crack movement at pier ‘L’ – shape and ‘T’-shape. From the figures, it can be concluded that cracks at Pier 33, P 25, P 15B, P 14B, P 13B, P 10A, P 11A, and P 12A are active cracks.