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Non-destructive evaluation of a city roadway for pavement rehabilitation: A case study

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Abstract

In this study, a pavement evaluation was carried out on a corridor that carried heavy truck traffic, connecting a harbor, a major highway, an airport, and industries. The pavement of the corridor was distressed frequently, requiring constant maintenance. With tight maintenance budget and intention to minimize the effects on traffic, full reconstruction or major rehabilitation of the corridor was not practical. To develop proper pavement rehabilitation design within the practical constrains, pavement evaluation was performed in this study using non-destructive techniques, such as visual condition survey, falling weight deflectometer (FWD), ground penetrating radar (GPR), and dynamic cone penetrometer (DCP), and pavement coring. It is demonstrated that appropriate pavement rehabilitation design can be developed to offer practical solution to the repair/maintenance for city streets.

Keywords: FWD; Deflection; Resilient modulus; DCP; GPR

1. Introduction and objectives

Many asphalt city streets/highways in Taiwan are subjected to heavy traffic, resulting in rapid deterioration that requires frequent rehabilitation. Because of the heavy traffic, rehabilitation of distressed city pavements is generally limited to milling and overlay. However, without proper understanding of the causes of the distresses, it has been often that the real deficiencies were not corrected and similar distresses recur within a short period of time.

To address these recurring issues, a comprehensive pavement evaluation, using various non-destructive techniques and pavement coring, were carried out on a city highway, Zhongshan Road located in Xiaogang Dist., Kaohsiung City. The various techniques used in the project included falling weight deflectometer (FWD) testing, ground penetrating radar (GPR) testing, dynamic cone penetrometer (DCP) testing, visual pavement condition survey, and pavement coring. Laboratory testing was also performed. The purpose of this study was to demonstrate the use of a compressive pavement evaluation for proper pavement rehabilitation design through a case study.

2. Literature review

Falling weight deflector (FWD) has been used successfully worldwide to characterize the conditions of pavement structures. Yuan and Shan [1] used a dynamic infinite element method to assess the dynamic response of flexible

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pavement under FWD loading. They suggested that FWD technique can be applied to assess conditions of flexible pavement under actual dynamic loads. Cheng and Zhang [2] pointed out that FWD should be broadly applied to the pavement evaluation because it is a non-destructive testing technique with adequate accuracy, reliability, convenience, and safety. Although FWD has been used for many decades, different researchers [3–5] had proposed different back-calculation methodologies that would yield significant different results.

Wang [6] proposed a simple analytical approach by including the classical solution of a one-dimensional heat equation in a homogeneous half-space and a Gaussian quadrature numerical scheme to promptly predict the transient temperature profile of a multilayered flexible pavement. He showed that the analytical approach can help field engineers rapidly assess the temperature profile of a multilayered flexible pavement according to pavement surface temperatures measured during FWD testing. Shen [7] applied the FWD evaluation technique to investigate the effects of seriflux injection on pavement. He noticed that the seriflux injection could effectively reduce the bending of pavement surface, increase the modulus of base or subbase layer, and decrease the pavement surface permeability. Loizos et al. [8] studied the efficiency of the FWD on simulating the traffic loading in recycled pavement. They conducted an in situ experiment to measure and record strain developments under both FWD loading and truck loading. They found that the maximum tensile strains obtained from FWD loading were very close to the values generated by the traffic loading.

Stehlik et al. [9] conducted a study evaluating the use of waste building materials for pavement construction. A full-scale pavement test section was built and was subjected to dynamic cyclic loading. The well-optimized cement-bound granular mixtures (CBGMs) were obtained by functional laboratory analyses with tests performed in real conditions on the testing section. The elastic moduli (E) of the CBGMs were obtained from the laboratory using a cyclic triaxial device. FWD testing was also performed on the test section and elastic moduli for CBGMs were back-calculated. A relationship between E determined by the cyclic triaxial test and back-calculated from the FWD was proposed as $E = 23\ \mathrm{Mr}^{0{,}91}$.

Shafiee et al. [10] applied FWD tests to evaluate seasonal influence on the Hot Mix Asphalt (HMA) pavements. They concluded that the vertical stress in unbounded layers and horizontal strain in HMA layer could be overestimated and vertical strain might be under-estimated. Varma and Kutay [11] developed a genetic algorithm-based back-calculation scheme using FWD results at different temperatures to estimate viscoelastic and nonlinear flexible pavement layer properties. The scheme was verified with field measured deflection data. Zhao et al. [12] used spectral element method (SEM) to dynamically back-calculate the properties of asphalt pavement layers subjected to FWD loadings. To simulate the characteristics of viscoelastic and damping behavior of AC, the modified Havriliak–Negami mode was applied. It was concluded that the surface deflection of AC pavement was greatly affected by the inertia and damping behavior when it is subjected to FWD loadings, especially for pavement base layers stabilized with cement. The dynamic influences should be considered during interpreting FWD data.

Pożarycki [13] proposed possible practical applications of numerical simulation for plate bearing test on asphalt pavement based on FWD deflection data. The simulated results of the second load–displacement curves were consistently close to the in situ plate bearing test results for base or subgrade layers of pavement.

Hu et al. [14] applied ground penetrating radar (GPR), dynamic cone penetrometer (DCP), magnetic imaging tomography (MIT), and dielectric measurement to estimate thicknesses of pavement and stabilized subgrade layers both in the field and laboratory. Thicknesses of asphalt layer were estimated by GPR and MIT testing and the results were compared with those obtained from core samples. It was observed that, when dielectric gauge values were applied, the average error was about 11% for the asphalt thicknesses estimated by GPR testing, but decreased to about 4% when calibrated with core samples. Results obtained from MIT were about 9% higher than those obtained from GPR.

With GPR and FWD testing, Ahmed and Tarefder [15] used a Mechanistic-Empirical design methodology to predict pavement quality/performance. Three different locations were tested. It was concluded that, with GPR testing, the thickness of AC layer was more consistently predicted than that of the base layer. Moreover, the back-calculated elastic moduli from FWD deflections exhibited differences at different locations. To collect information of underground layers of pavement, Dong et al. [16] developed a vehicle-mounted GPR detection system by combining a horizontal high-pass filter and a modified layer localization method.

3. Non-destructive pavement testing program

As mentioned in Section 1, a deteriorated city street pavement was investigated in this study. Shown in Fig. 1, the experimental section was located at Zhongshan Rd., Xiaogang Dist., Kaohsiung City in Taiwan, with a total length approximately 12,800 m. This roadway was a corridor connecting industrial parks and a harbor area, carrying heavy truck traffic, primarily container trucks. Original pavement was constructed many years ago and the pavement had gone through rehabilitation multiple times; therefore, construction records were not readily available and the original pavement structure and material properties were not known.

Based on 98-day consecutive recorded information provided by the Public Works Bureau of Kaohsiung City Government, the average daily traffic (ADT) in Zhongshan

Fig. 1. The experimental section at Zhongshan Road.

Road was 25,000, with 50% container truck traffic. The estimated 20-year 80kN Equivalent Single Axle Load (ESAL) was approximately 89 million. Pavement temperature could reach 65 °C during the summer seasons. High traffic loads and temperatures resulted in severe distresses requiring frequent repairs.

Pavement evaluation of the test road included various non-destructive testing and pavement coring, including the following:

- Visual condition surveys using Pavement Condition Index (PCI)
- Falling weight deflectometer (FWD) testing
- Ground penetrating radar (GPR) testing
- Dynamic cone penetrometer (DCP) testing
- Pavement coring

3.1. Visual pavement condition survey

Detailed visual condition surveys were performed on Zhongshan Road following the ASTM Standard D6433 – Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys. Results of the sixty-four (64) survey samples are shown in Table 1. As can be seen from Table 1, PCI values range from 26 to 34 and all are classified as in poor condition. Alligator cracking and rutting were the dominant distresses observed in the field. Fig. 2 shows the typical distresses observed in the field. Cracks with width of more than 19 mm were observed. From visual inspection, it was estimated that more than 10% of aggregates might have been lost from the asphalt mix. High severity rutting, with depth greater than 19 mm, were noted on the pavement section.

Table 1
Summary of PCI values.

PCI values	26	30	31	32	34
Number of sample sets	1	4	1	8	49
PCI category	Poor	Poor	Poor	Poor	Poor
PCI average	33.1				
PCI standard deviation	±2.1				

(a) rutting (b) cracking

Fig. 2. Typical pavement distresses (a) rutting, and (b) cracking.

Pot holes were another type of distress commonly seen on the pavement. In some portions of the roadway, the depths of pot holes were larger than 50 mm and frequent patch repairs were required. Direct overlay was the common rehabilitation method used in Kaohsiung City. The added asphalt layer could not provide stuffiest strength to sustain the heavy truck loadings. As a result, sever shoving (about 50 mm thick) was noticed along pavement edges.

3.2. Falling weight deflectometer (FWD) testing

FWD deflection testing was performed on the pavement section using a Dynatest 8000 device, as seen in Fig. 3. In Taiwan, the regular interval in FWD testing on highway was between 300 and 500 m. However, due to the frequent changes in pavement structures for city streets, the FWD testing was performed at 200 m interval in this evaluation. The roadway consisted of multiple lanes in each traffic direction and testing was conducted on the outside lanes (travel lane). The FWD had seven (7) sensors spaced at 30 cm on center.

FWD testing was performed to assess the structural condition of the test sections. In addition, deflection data were analyzed to back-calculate elastic moduli of pavement layers, using the back-calculation software MODULUS version 6.0, developed by the Texas Transportation Institute [17,18].

3.3. Ground penetrating radar (GPR) testing

As mentioned earlier, construction and maintenance records were not readily available and pavement structures and material data were not known. GPR testing was done to help estimate pavement layer thickness; to identify possible underline structures (pipes); and to assess the uniformity of the pavement structure. Two types of GPR testing devices were employed in this study. Model SIR-20 GPR manufactured by Geophysical Survey System, with 400 MHz antenna, was used to conduct testing in transverse direction (perpendicular to traffic direction). Locations of the testing are marked as yellow stars on Fig. 1. Fig. 4 shows the GPR testing in an intersection. GPR testing along the longitudinal direction of the roadway was also performed using a GPR devise with a 1500 MHz antenna. Testing was conducted along outside lanes on both directions.

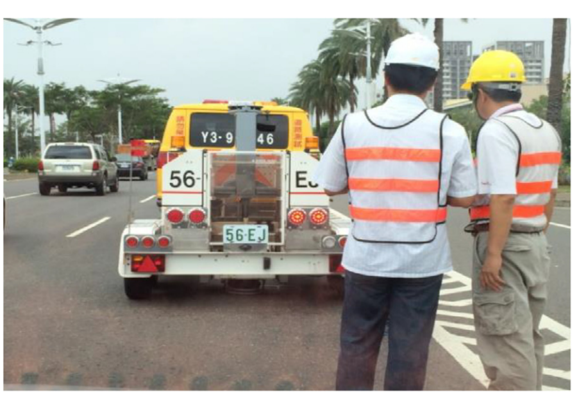


Fig. 3. Dynatest 8000 FWD device.

Fig. 4. GPR testing in transverse direction with 400 MHz antenna.

Fig. 5. In situ DCP testing.

3.4. Dynamic cone penetrometer (DCP) testing

To increase the accuracy in estimating the thicknesses of the base and subbase layers, DCP testing was conducted in this study. DCP could also be used in evaluating the strength of the base/subbase layers. The yellow stars on Fig. 1 also indicate the locations of the DCP testing. An in situ DCP testing is shown in Fig. 5.

DCP was invented by Scala in 1956 and is also known as Scala penetrometer. DCP is a hand-held instrument that can easily penetrate base/subbase and subgrade soils. The design of DCP was changed by Kleyn to extend its engineering application to assess the stiffness of the pavement layers. In DCP testing, ratio of penetration depth to accumulated blow number is defined as penetration ratio (PR) or dynamic cone penetrometer index (DCPI). DCPI had been used in estimating in situ California bearing ratio (CBR) and resilient modulus (MR).

The testing followed the ASTM standard D6951-09, Standard Test Method for Use of the dynamic cone penetrometer in Shallow Pavement Applications. In general, a DCPI less than 7 mm/blow was required for base/subbase courses containing granular aggregates; and less than 25 mm/blow for base/subbase courses containing silty or clay soil [19].

3.5. Pavement coring

Twenty-two (22) Pavement cores were obtained in this study to evaluate the condition of the asphalt layer and to provide accurate determination of the thickness of the asphalt layer. This information was used in assisting the FWD data analysis.

3.6. A case study: evaluation of a pavement rehabilitation methodology

The routine pavement maintenance used by Kaohsiung City government was to mill 5–10 cm of AC surface layer and replaced with the same type and thickness of the AC layer. Evaluation was seldom performed before the pavement rehabilitation. In this study, evaluation of a 2200 m segment of the Zhongshan Road, located at the south end of the roadway (from 9600 m to 11,800 m), was carried out. Location of this segment is shown in Fig. 1 in green color.

The Kaohsiung City government had good experience in using asphalt concrete containing basic oxygen furnace (BOF) slag as part of coarse aggregates. For this 2200 m section, 10 cm of existing AC layer was milled out and overlaid with 10 cm of AC with BOF slag. The IV-C dense graded was used as the basis for the BOF slag mix design. The design was based on the AI MS-2 standard, in the design, 40% of coarse aggregates were replaced by BOF slag. Because of the characteristic of BOF slag, the optimum asphalt content in the mix was 4.0% by weight, less than typical dense graded asphalt concrete mix in Taiwan.

4. Results and discussion

4.1. Pavement coring

Twenty-two (22) core samples were obtained from the test pavement section. The purposes of the coring were to visually asses the asphalt pavement condition and to determine the AC layer thickness. Typical condition of the cores is depicted in Fig. 6. As indicated earlier, the pavement had gone through several maintenance and rehabilitation activities. Three distinct layers can be identified. In conjunction with DCP testing and GPR testing, measured core thicknesses were used in determining the pavement layer thicknesses. Accurate thicknesses were required in FWD analysis.

4.2. DCP test

DCP testing was performed on the outside lanes on both the southbound and northbound directions. Test locations are represented as yellow stars on Fig. 1. DCP test results were used in distinguishing the base and subbase layers and in determining their thicknesses. A typical DCPI results obtained from northbound lane is shown in Fig. 7. As can be seen from the figure, the DCPI value started to change at the depth of 40 cm, indicating the change of materials. Therefore, it could be inferred that the boundary between the base and subbase layers was located at a depth of 40 cm. From the test results, thicknesses of the base and subbase layers were estimated to be 40 cm and 35 cm, respectively.

4.3. GPR testing

The GPR devise with a 400 MHz antenna was applied along transverse direction of the pavement at the locations indicated in Fig. 1 (yellow stars). The consecutive direct wave signal indicated the depth (or thickness) of a certain layer. A flat direct wave generally indicates a uniform layer. Obstacles might exist in a layer when wavy-shaped signals were observed. The devise can provide information to the maximum depth of 2.5 m.

A typical GPR output is shown in Fig. 8. As shown in the figure, the 0.0–0.1 m was the depth between GPR and ground level (air) and was not included in this analysis. Between the depths of 0.1–0.3 m, the color was consistent and the separation between direct waves was apparent. This should be the AC surface layer. Similarly, the depth

(a) Core Location (b) Typical Core Sample

Fig. 6. Typical condition of pavement core sample.

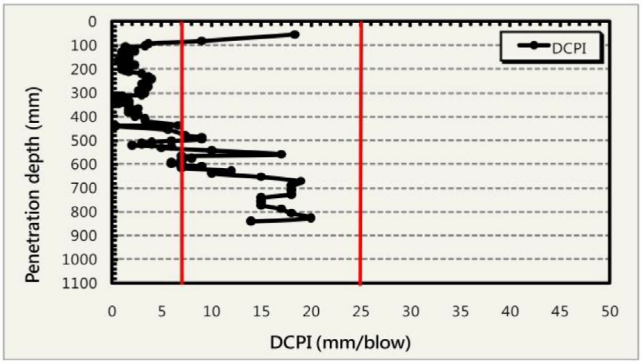


Fig. 7. A typical DCPI graph from northbound travel lane.

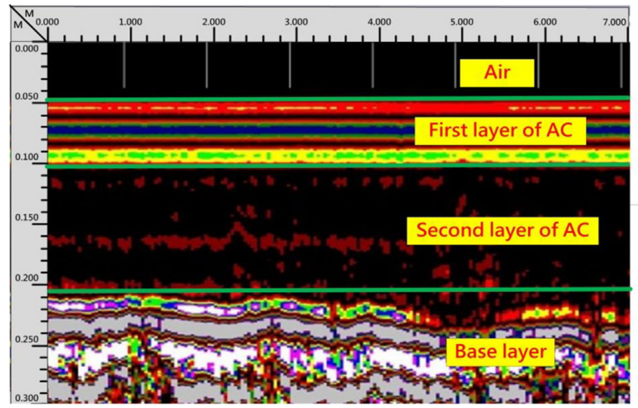


Fig. 9. A typical GPR output with a 1500 MHz antenna.

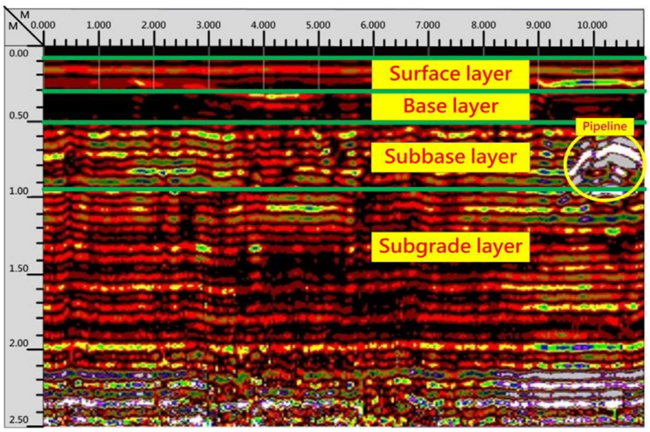


Fig. 8. A typical GPR output with 400 MHz antenna.

from 0.3 m to 0.5 m was determined to be the base layer. The subbase layer was located from 0.5 to 1.2 m.

Also shown in Fig. 8 are some white wave peaks. This is an indication that a pipeline might be located at the depth about 0.7 m. Therefore, GPR devise with 400 MHz antenna might be used for pipeline detection under the pavement structure.

A GPR devise with a 1500 MHz antenna was also used in the study along the longitudinal direction (traffic direction) of the roadway. The maximum depth detected by the 1500 MHz antenna was approximately 0.5 m. Although, the evaluation depth is shallower, the device can provide more accurate measurements. A typical output is shown in Fig. 9. The depth from 0 to 0.05 m represents the distance between the GPR device and the pavement surface and was excluded from analysis. From the figure, it can be seen that, at depths from 0.05 to 0.10 m, the wave signal was flat and straight, indicating a uniform layer. At the depth of 0.10–0.20 m, the wave signals were quite consistent. However, no apparent reflection changes were observed when the wave reached the depth of 0.2 m. It indicated that the AC between the depths of 0.1–0.2 m was probably distressed.

4.4. Determination of in situ pavement layer thickness

To perform pavement evaluation properly, accurate layer thicknesses are required. Data obtained from the pavement coring, DCP testing and GPR testing were analyzed to determine the existing pavement layer thicknesses, as described in previous sections. A summary of the layer thicknesses at various locations are presented in Table 2.

Fig. 10 shows the comparison of the AC thickness obtained from core samples and GPR data. As shown in Fig. 10, little difference exists between these two sets of data. Thicknesses obtained from GPR, in conjunction with the DCP testing, were used in the FWD data analysis since they cover the entire roadway and have been calibrated by the core sample thicknesses.

4.5. FWD test data analysis

As mentioned in a previous section, FWD tests were performed at a 200 m interval along the outside lane of the Zhongshan Road. Fig. 11 shows the deflections under the center of the loading plate of FWD obtained from northbound direction. From past experiences observed on roadway pavements in Kaohsiung City, the measured deflections directly under loading need to be below 0.635 mm. A deflection higher than this value indicted an insufficient bearing capacity. As can be observed from Fig. 11, several locations on the roadway exhibited deflections greater than this limiting value. They were located at 600, 4600, 5880, 5960, 10,200, 10,600, and 12,000 m.

Based on the results obtained from cores and DCP tests of the north-bound lane, the thicknesses of the surface, base, and subbase layers used in FWD deflection data analysis were 19.1, 39.3, and 35.0 cm, respectively. Again, using the measured FWD deflections from northbound lane (outside lane), elastic moduli for AC layer (E1), base course (E2), subbase course (E3) and subgrade (E4) were back-calculated using MODULUS 6.0 software. The back-calculated elastic moduli for pavement on the outside lane of northbound direction are presented in Fig. 12.

Table 2
Summary of pavement layer thicknesses.

Location	Core sample thickness (cm)	Existing AC thickness (cm)		Base thickness (cm)	Subbase thickness (cm)
		First layer	Second layer		
300	17.6	6	12	38	30
600	17.7	5	13	37	45
1000	18.8	6	12	38	25
2800	21.8	5	15	39	50
3700	18.8	5	13	45	30
4600	19.8	6	14	40	60
5600	17.9	5	13	40	60
6000	20.3	5	16	38	25

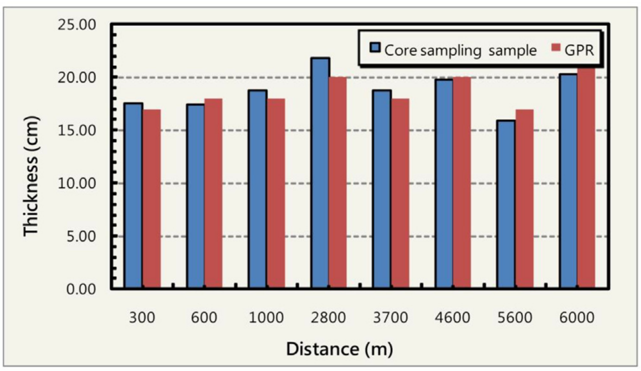


Fig. 10. Comparison of the AC thickness obtained from core samples and GPR data.

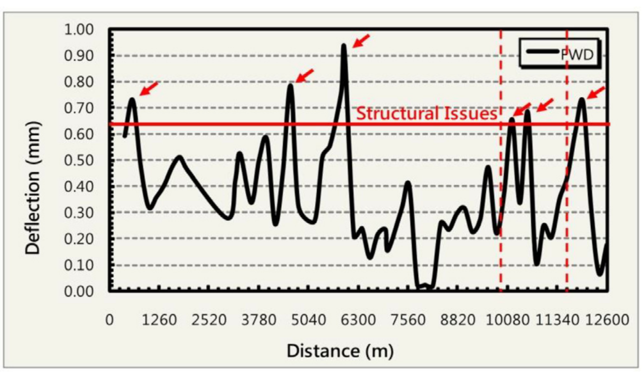


Fig. 11. FWD Deflections under loading center – northbound direction.

Based on many years of field experiences in Kaohsiung City, typical values of E1, E2, E3, and E4 should be above 14,060, 703, 703, and 562 kg/cm^2, respectively, to indicate sufficient pavement structural capacity. As shown in Fig. 12, E1 on many locations was less than the limit (14,060 kg/cm^2), indicating that the AC layer on those locations was structurally deficient. Similar results were observed for E2 values. For E3 and E4, the back-calculated values were generally greater than the limiting values. The back-calculated elastic moduli, E1, E2 and E3 were used by Kaohsiung City Engineering department in determining the needs for pavement rehabilitation.

The routine flexible pavement maintenance used in Kaohsiung City included milling 5 cm of AC and overlaying with 5 cm of new AC layer. In this study, the rehabilitated pavement section was considered to have two AC layers, the 5 cm of AC overlay and the remaining existing AC layer. The original and the rehabilitated pavement structures are presented in Fig. 13, with corresponding thicknesses and elastic moduli. Please note that the base and subbase courses were combined into one single layer for this analysis. As indicated in the figure, the layer thicknesses are 5.0 cm, 14.1 cm, and 74.4 cm.

Again, MODULUS 6.0 program was used in back-calculating the layer elastic moduli for the rehabilitated pavement structure. The back-calculated elastic moduli are presented in Fig. 14. From this figure, it can be observed that the E1 and E2 values on many locations are less than the minimum value of 14,060 kg/cm^2, indicating insufficient structural capacity. It can therefore be inferred that the routine maintenance strategy used in Kaohsiung City was not adequate in providing sufficient pavement structural strength.

4.6. Evaluation of a pavement rehabilitation methodology – a case study

4.6.1. Conventional rehabilitation methodology

A 2200 m section of Zhongshan Road, located between 9600 m and 11,800 m (south end), was evaluated in this study to determine if the conventional rehabilitation method was adequate. The AC surface was milled to a depth of 10 cm and overlaid with the same thickness of BOF slag AC surface. After open to traffic for half a year, cracking and yellow color mud slurry were observed on part of rehabilitated pavement surface, as shown in Fig. 15.

Additional distresses were observed on other locations, at 10,200, 10,600, and 11,800 m after nine (9) months of traffic, as depicted in Fig. 16. Major distresses were alligator cracking and rutting (1–2 cm depth). To understand the causes of these distresses, FWD and DCP testing were conducted on the distressed locations.

DCP tests were performed on the rehabilitated section at locations of 10,200 and 10,600 m, and the results are shown in the Fig. 17. As seen in the figure, the DCPI values at both depth of 0–9 and 50–85 cm were greater than 7,

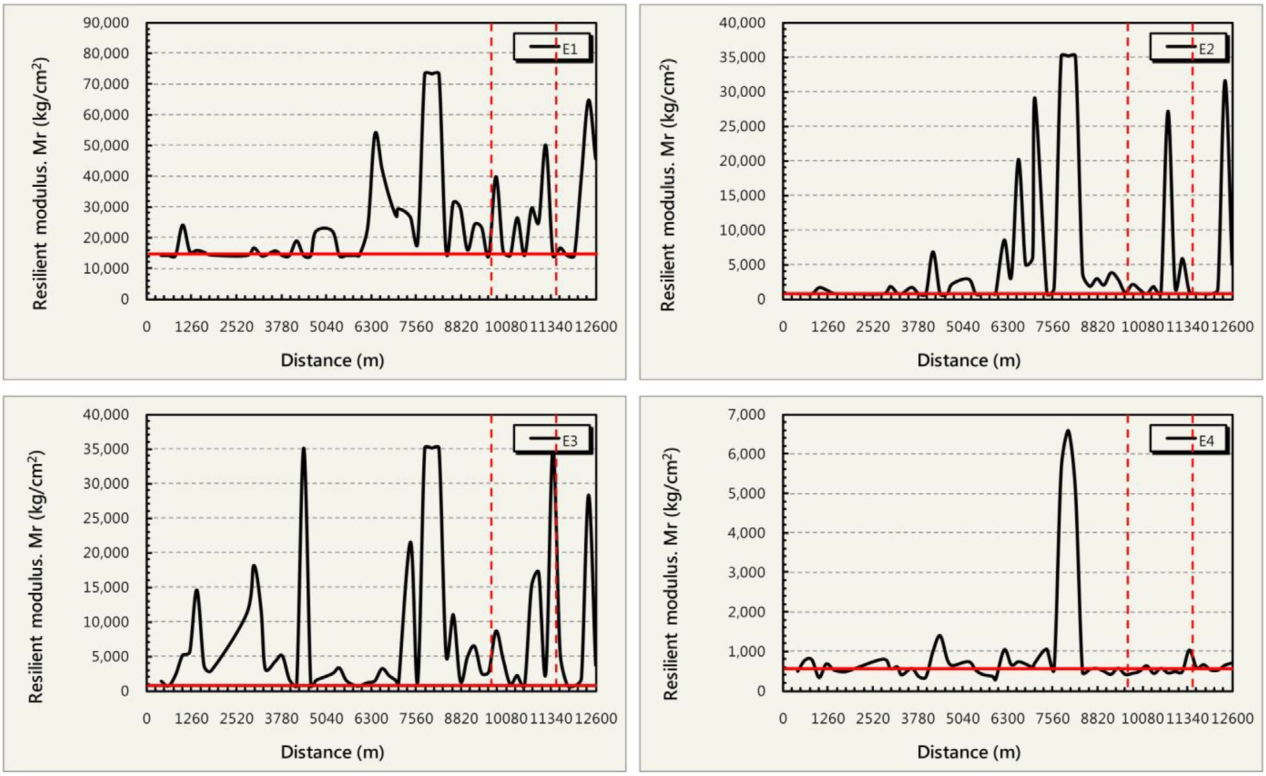


Fig. 12. FWD Back-calculated elastic moduli of pavement layers – northbound direction.

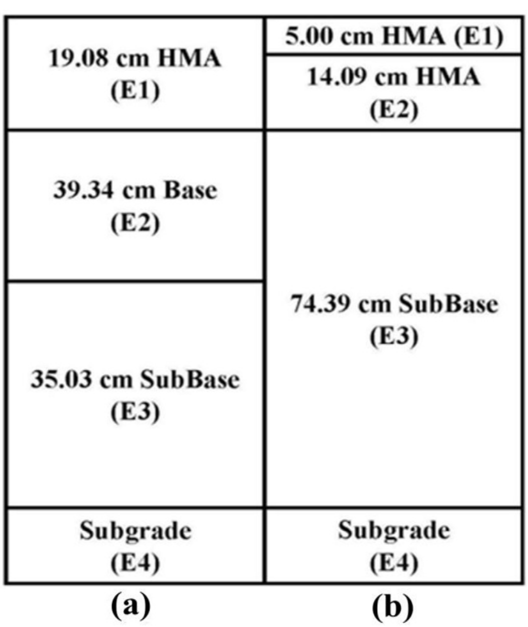


Fig. 13. Pavement structure – (a) existing pavement and (b) rehabilitated pavement.

which were considered deficit for base/subbase courses. The insufficient structural capacity was further manifested by the extremely heavy traffic volume with container truck loadings on the pavement.

Fig. 18 shows the maximum deflections under FWD loading center measured at different locations on the rehabilitated BOF slag AC pavement. As shown in the figure, the deflections at locations of 10,200, 10,600, and 11,800 m were greater than the limiting value of 0.635 mm, indicating insufficient pavement structure capacity for the traffic loading.

FWD deflections were also used to back-calculate the pavement layer elastic moduli at the locations with distresses. The estimated elastic moduli for the various layers at 10,600 m are shown in Fig. 19. The estimated elastic moduli for E1, E2, E3, and E4 were 14,062, 703, 703, and 581 kg/cm^2, respectively. Comparing to the critical values, these values were considered marginal and might not be sufficient to sustain the heavy traffic volume experience on this pavement. Similar results were observed at the locations of 10,200 and 11,800 m. Therefore, the conventional rehabilitation is not considered adequate.

From the evaluation, it can be stated that the non-destructive testing techniques, FWD, DCP and GPR, can be used to perform pavement evaluation and develop pavement rehabilitation design. It is also recommended that pavement evaluation be conducted as the basis of selecting appropriate pavement rehabilitation methods.

4.6.2. Development of pavement rehabilitation design for the test section

In the previous section, it was concluded that the conventional pavement rehabilitation method was not appropriate for Zhongshan Road pavement. Efforts were made in this study to assess the effectiveness of six (6) different

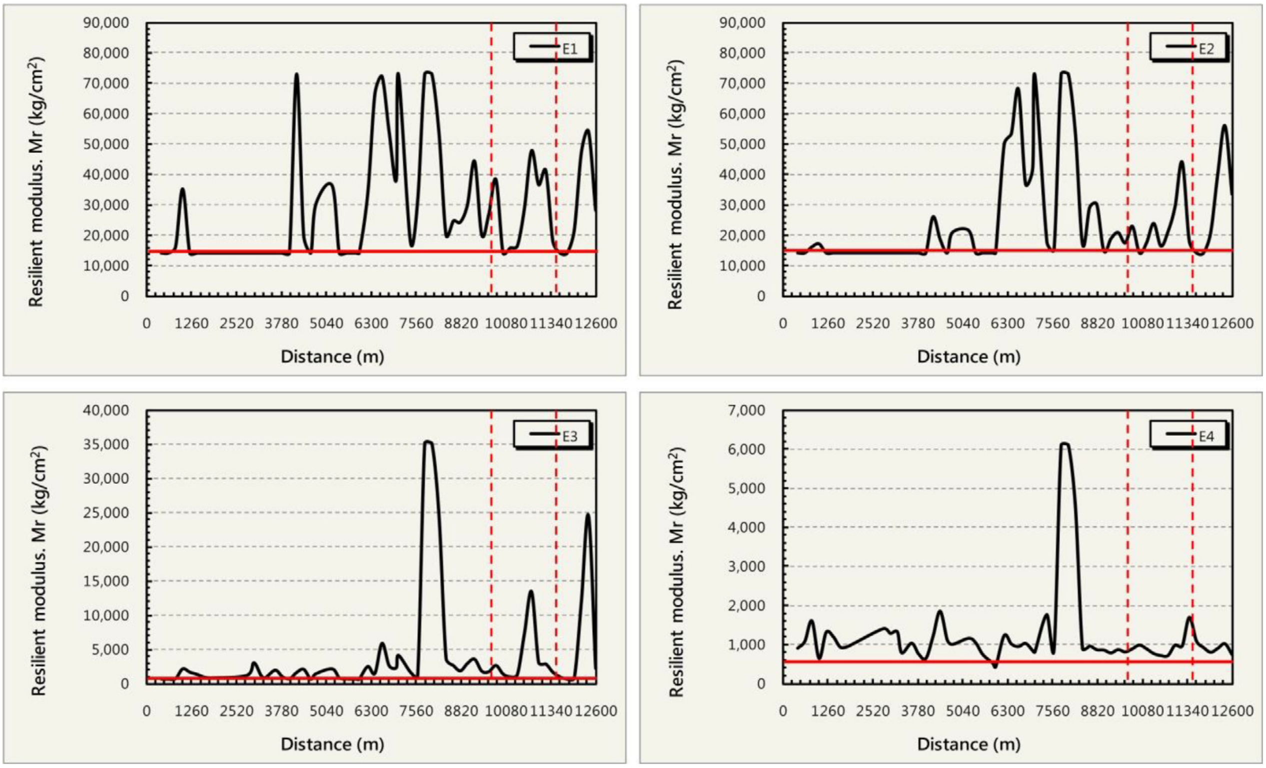


Fig. 14. Back-calculated elastic moduli for routinely rehabilitated pavement – northbound direction.

Fig. 15. Pavement distress at location of 10,600 m.

rehabilitation alternatives. The six pavement rehabilitation design alternatives are summarized below.

Case I

- Known or assumed parameters
  - o Design traffic – 10,000,000 ESALs
  - o E1 = 35,151; E2 = 28,191; E3 = 4,956; E4 = 792 kg/cm^2
  - o Existing AC layer thickness = 17.8 cm
  - o Existing combined base/subbase thickness = 76.2 cm
- Required pavement rehabilitation design
  - o Milling 10.2 cm of AC
  - o Overlaying 10.2 cm of AC to ensure the value of E1 of 35,151 kg/cm^2

Case II

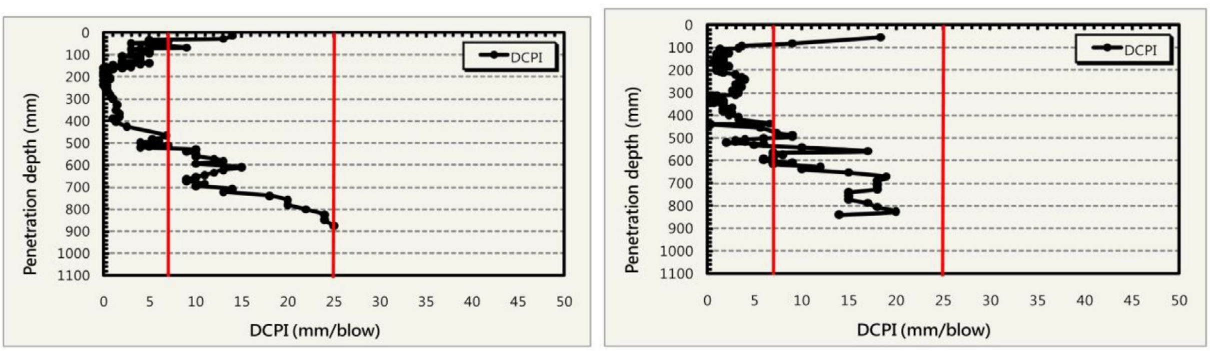
- Known or assumed parameters
  - o Design traffic – 20,000,000 ESALs
  - o E1 = 35,151; E2 = 28,191; E3 = 4,956; E4 = 759 kg/cm^2
  - o Existing AC layer thickness = 17.8 cm
  - o Existing combined base/subbase thickness = 76.2 cm
- Required pavement rehabilitation design
  - o Milling 10.2 cm of AC
  - o Overlaying 14.0 cm of AC to ensure the value of E1 of 35,151 kg/cm^2

Case III

- Known or assumed parameters
  - o Design traffic – 10,000,000 ESALs
  - o E1 = 35,151; E2 = 35,151; E3 = 703; E4 = 415 kg/cm^2

(a) 10200m (b) 10600m (c) 11800m

Fig. 16. Distress observed at 9 month, at (a) 10,200, (b) 10,600, and (c) 11,800 m.



(a) 10200m (b) 10600m

Fig. 17. DCP tests performed at locations of (a) 10,200 and (b) 10,600.

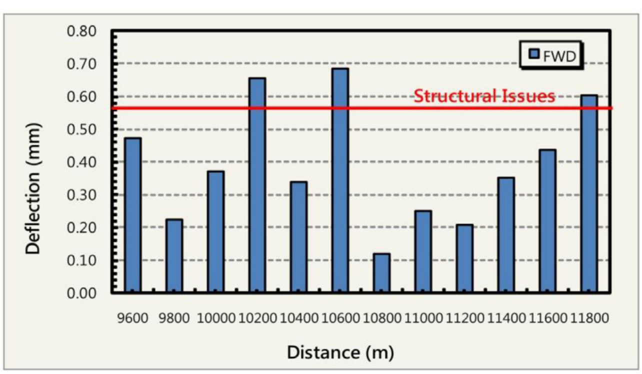


Fig. 18. Deflections measured under center of FWD loading plate at different locations.

- o Existing AC layer thickness = 17.8 cm
- o Existing combined base/subbase thickness = 76.2 cm

- Required pavement rehabilitation design
  - o Milling the entire depth of AC (17.8 cm)
  - o Remove base layer
  - o Overlaying 22.8 cm of AC, with no base course, to ensure the value of E1 of 35,151 kg/cm^2

Case IV

- Known or assumed parameters
  - o Design traffic – 10,000,000 ESALs
  - o E1 = 35,151; E2 = 14,060; E3 = 703; E4 = 415 kg/cm^2
  - o Existing AC layer thickness = 17.8 cm
  - o Existing combined base/subbase thickness = 76.2 cm

- Required pavement rehabilitation design
  - o No milling of AC
  - o Overlaying 17.8 cm of AC to ensure the value of E1 of 35,151 kg/cm^2

Case V

- Known or assumed parameters
  - o Design traffic – 20,000,000 ESALs
  - o E1 = 35,151; E2 = 14,060; E3 = 703; E4 = 415 kg/cm^2
  - o Existing AC layer thickness = 17.8 cm
  - o Existing combined base/subbase thickness = 76.2 cm

- Required pavement rehabilitation design
  - o Milling the entire depth of AC (17.8 cm)
  - o Remove the base course

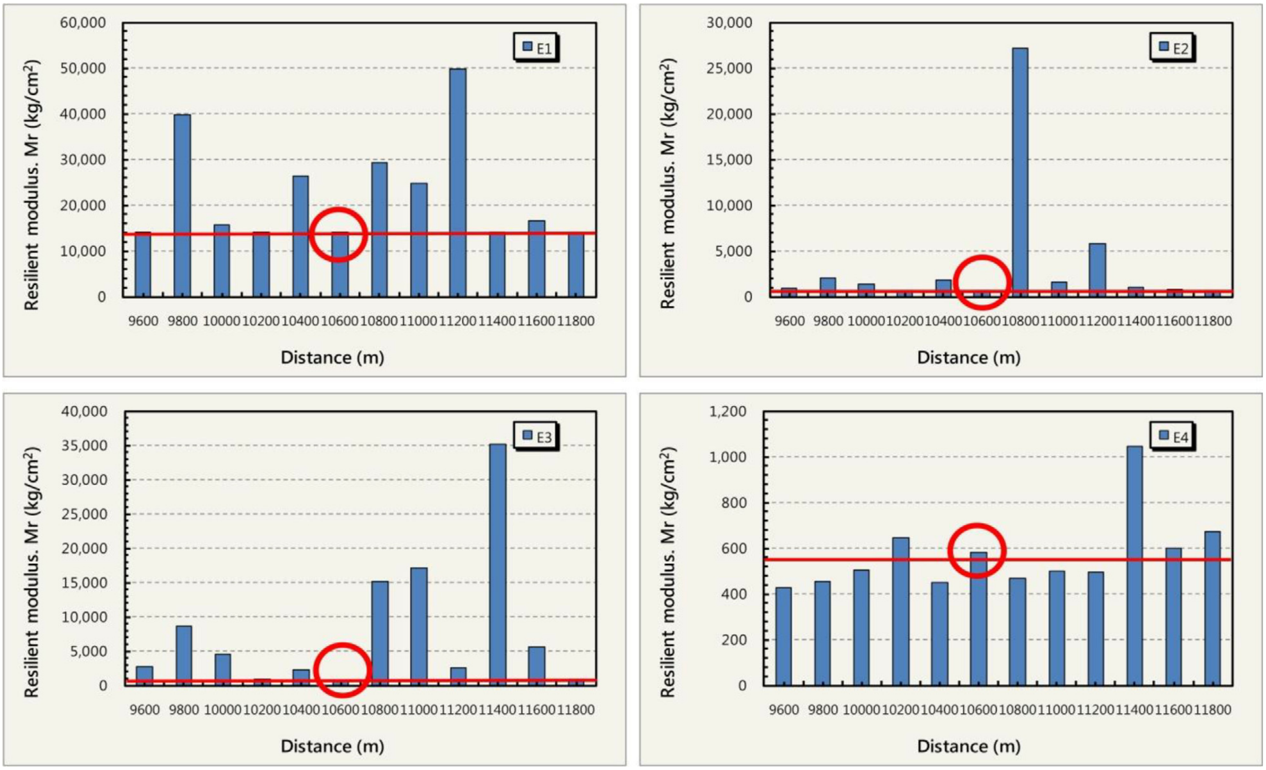


Fig. 19. The FWD back-calculated elastic moduli for the north-bound direction.

- o Overlaying 25.4 cm of AC, without base course, to ensure the value of E1 of 35,151 kg/cm^2

Case VI

- Known or assumed parameters
  - o Design traffic – 20,000,000 ESALs
  - o E1 = 35,151; E2 = 14,060; E3 = 703; E4 = 415 kg/cm^2
  - o Existing AC layer thickness = 17.8 cm
  - o Existing combined base/subbase thickness = 76.2 cm

- Required pavement rehabilitation design
  - o No milling of AC
  - o Overlaying 22.9 cm of AC to ensure the value of E1 of 35,151 kg/cm^2

From these case studies, it is shown that, without improving the base/subbase courses, the design pavement rehabilitation consisted of removing 10.2 cm of AC and replacing it with 10.2 cm of AC overlay. However, with the design traffic doubled (from 10 million to 20 million ESALs), the design overlay became 14.0 cm.

5. Conclusion

In this study, non-destructive testing techniques, including visual condition survey, FWD, GPR and DCP, were used in evaluation of Zhongshan Road pavement. Zhongshan Road is a busy street, connecting the industrial area and port. Pavement rehabilitation designs were developed based on the pavement evaluation. From the study, the following conclusions are drawn:

1. The thickness of pavement layers can be accurately estimated using the combined testing of GPR, DCP and pavement coring.
2. DCP testing can be used in assessing the strength of base/subbase courses. It can also be used in delineate the boundary of the base and subbase layers.
3. FWD deflections were used, in conjunction with the MODULUS 6.0 software, to evaluate the strength of the pavement structures. From the evaluation, structural deficiencies were observed at various locations of Zhongshan Road.
4. The case study illustrated that proper pavement evaluation is essential in selecting the appropriate pavement rehabilitation alternatives for deteriorated pavement.

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