USE OF THE BENKELMAN BEAM IN DESIGN AND CONSTRUCTION OF HIGHWAYS OVER SOFT CLAYS

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The Benkelman beam was used for the pavement design of a highway in Thailand over soft, deltaic clay soils. Because little technical information on the validity of this technique over soft clays became available, research into testing techniques was carried out. It was found that a seasonal deflection effect occurred, that the temperature correction factor is less in tropical countries than in temperate climates, that the movement of the front leg of the Benkelman beam was significant over soft clay subgrades, and that the Canadian test procedure was preferable to the Western Association of State Highway Officials test procedure. It was concluded that design deflection criteria established by Canadian and British pavement trials were applicable to highways over soft clay subgrades, and design deflections for both the Canadian and British Benkelman beam testing procedures were determined. Benkelman beam tests were carried out during construction to check that these values were obtained, and they indicated that practically all readings fell below the design limits. The constructed pavement was therefore considered to be adequate. Tests on various pavement layers indicated that the crushed rock base course was the most effective material in reducing deflections relative to its cost over these soft clays, and this resulted in a pavement redesign that resulted in construction cost savings of $700,000.

•THE pavement rebound deflection, as measured by the Benkelman beam under a standard wheel load, has been found to correlate extremely well with the subsequent life of the roadway pavement (1). The reason for this correlation is that pavements with larger rebound deflections are likely, under repeated loads, to exceed certain critical strains that lead to permanent strains and consequent pavement distress.

The major technical development of this pavement testing procedure has occurred in three geographical areas: in California by the California Division of Highways (2), in Canada by the Canadian Good Roads Association (CGRA) (3), and in England by the U.K. Transport and Road Research Laboratory (TRRL) (1,4). TRRL has also carried out some worthwhile development work in tropical countries such as Malaysia and Ghana (5).

It was decided to use the Benkelman beam in the design and construction of the 53-mile-long (85-km) Thon Buri-Pak Tho Highway in Thailand, which traverses soft, swampy, deltaic clay soils for its full length. The California bearing ratio (CBR) of the soft clay subgrade was always less than 2, and embankment settlements ranging from 8 to 40 in. (20 to 100 cm) were anticipated. The CBR pavement design method was not considered to be a reliable method at these low CBR values, and a more fundamental approach using pavement rebound deflections was considered to be more reliable under such difficult subgrade conditions.

There is little technical literature concerning the use of Benkelman beam techniques over soft clay subgrades (6), and considerable development work had to be carried out

Publication of this paper sponsored by Committee on Pavement Condition Evaluation.

on testing procedures, temperature correction factors, and design deflection values before this pavement design method could be used with confidence in this situation. Pavement rebound tests were also carried out during construction of the embankment and pavement layers to check actual deflections against the required values. A considerable amount of technical information has been accumulated on the use of the Benkelman beam in pavement design over the soft soils found beneath this highway, and this is the subject of this paper.

TESTING PROCEDURES

Benkelman Beam Leg Movements

Several different testing procedures have come into being during the development of the Benkelman beam. The original test procedure introduced during the Western Association of State Highway Officials (WASHO) Road Test (7) has been adopted with slight modifications in both California and England; i.e., the test starts with the wheel load midway between the probe and the front leg of the Benkelman beam, and a transient maximum reverse reading and stationary final reading are taken as the wheel load moves forward.

Development work in Canada resulted in considerable modifications to the WASHO type of deflection test procedures. The axle load was increased from 14,000 to 18,000 lbf (62 to 80 kN), and transient deflection readings were eliminated in favor of three static readings over the test point, over a point 8 ft 10 in. (2.7 m) from the probe (this was equivalent to testing at the front legs of the beam), and at a point a considerable distance away. This testing procedure was developed to overcome one drawback in the WASHO procedure, i.e., that the maximum or reverse reading in this test (which occurs when the wheel load is over the probe point) is often influenced by the deflection of the front legs of the Benkelman beam apparatus. The Canadian method corrects this leg movement by determining a true rebound from the following formula (for a 2:1 lever arm ratio):

$$X_T = X_A + 2.91\,Y \qquad (1)$$

where

X_T = true rebound,
X_A = apparent rebound, and
Y = leg movement.

The derivation of this formula is given elsewhere (8).

Deflection of the front leg occurs when the deflection bowl has a greater sphere of influence than 8 ft 10 in. (2.7 m), and such a situation is common over soft clays and over cement-treated base courses, where there is a large difference in elastic modulus between pavement and subgrade in both cases. This is shown in Figure 1, which shows some typical influence profiles for various locations in the central deltaic area of Thailand. It is seen that deflections occur even when the wheel load is 8 ft 10 in. (2.7 m) away from the probe point, and conversely this means that the beam legs will move when the wheel load is over the probe point. This leg movement is higher on softer soil types where the total deflection is larger.

An analysis of all readings indicated that the leg movement increased almost linearly with apparent rebound for the asphalt concrete surfacing up to a maximum value of about 0.008 in. (0.2 mm) while leg movements of between 0.005 and 0.012 in. (0.13 to 0.3 mm) for the sand embankment and pavement layers occurred. It is apparent that the rebound measured in the WASHO test procedure will be considerably in error because of these

high values for the leg movement because no corrections for these movements are made in this method.

Comparisons Between WASHO and Canadian Test Procedures

A considerable number of deflection tests were carried out to determine the difference between the WASHO and Canadian test procedures to determine which test procedure should be used to determine deflections in this situation where soft clay subgrades were present. The standard CGRA method (3) was carried out at one point, and then a slightly modified WASHO method used at the same point. For a valid comparison with the Canadian test results, the axle load used in the WASHO test procedure was 18,000 lbf (80 kN), and the tire pressure was 80 psi (552 kPa). Moreover, the final reading was taken at 38 ft 10 in. (11.8 m) from the probe point and not at 25 ft (7.6 m) as in the Californian method or at 10 ft (3 m) as in the British method. A third test procedure that is sometimes used by the Thai Highways Department was carried out. This involved beginning the test only 8 in. (20 cm) from the probe point instead of at a distance of 4 ft (1.2 m) as in the standard WASHO test procedure. A temperature correction of 0.001 (35 - temperature C) was made to convert all rebound readings to a standard temperature of 35 C.

It is evident that the trials were not designed to give the difference in deflection between the standard testing procedures, which would have been considerable because of the different wheel loads, but were designed to detect any difference due to (a) static and moving deflection readings and (b) the starting position of the wheel load, which could affect the initial position of the front leg of the Benkelman beam.

In Figure 2 it is seen that the Thai test method gives 10 percent higher rebound values than the WASHO procedure. This could be rationalized by considering that the Thai method is virtually a static test because of the closeness of the initial wheel load to the probe point but that the WASHO method is a transient test involving a moving wheel load.

In Figure 3 it is seen that the Thai test method also gives higher rebound values than the apparent rebound in the Canadian method. The difference between the rebounds in the two methods does not increase with the rebound, however, as in Figure 2, but is more constant at about 0.001 in. (0.025 mm). This indicates that the lower reading for the Canadian apparent rebound could be due to the push-up of the probe between the static wheels due to heave of the asphalt concrete. This does not occur in the Thai method.

In Figure 4 there is little difference between the WASHO rebound and apparent rebound in the Canadian method, but this could be due to compensatory effects, i.e., the moving wheel loads and pavement push-up as described above. In Figures 3 and 4, however, it is clear that there is a major difference of 10 to 50 percent between the Canadian method and the other methods when the Canadian procedure of using the leg movement to correct apparent rebounds to true rebounds is used. This difference increases as the rebound value increases because it has been observed that the leg movement increases as the rebound value increases. Considerable error in the actual rebound value would therefore occur if only the standard WASHO test were used on this highway over soft soils; e.g., from Figure 4, it can be seen that the WASHO test would have given a value of 0.012 in. (0.3 mm) when the true rebound was actually 0.025 in. (0.64 mm). The British procedure in which the final reading is taken only 10 ft (3 m) from the probe would have led to an even greater underestimate of the true rebound because the final reading would still be within the deflection bowl.

Positive residuals of between 0.005 and 0.015 in. (0.13 and 0.38 mm) were found to occur in the WASHO test. Residuals are due to (a) permanent deformation of the pavement (causing negative residuals), (b) the leg of the beam being within the deflection bowl (causing positive residuals), and (c) the shading effect on the beam at the beginning of the test [causing positive residuals of 0.005 to 0.010 in. (0.12 to 0.25 mm) in tropical countries (9)]. Scala (10) has shown that the positive residuals measured in sound pavements with no shading effect should be twice the initial leg movement at the start of the test. No such relationship was found for the WASHO tests carried out on the

Figure 1. Influence profiles over soil types.

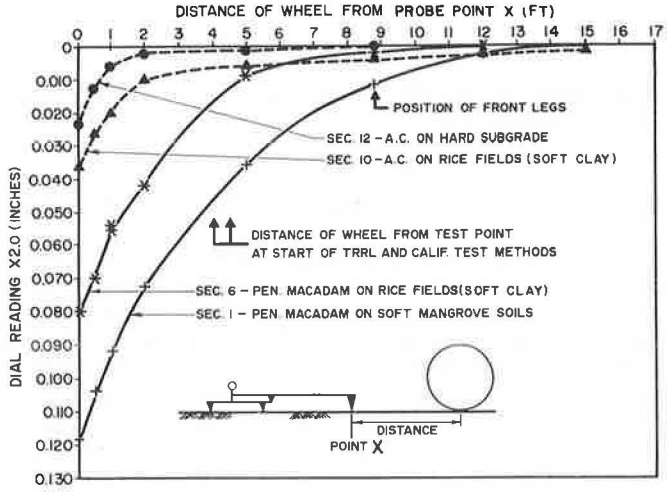


Figure 2. Correlation between Thai and WASHO test procedures.

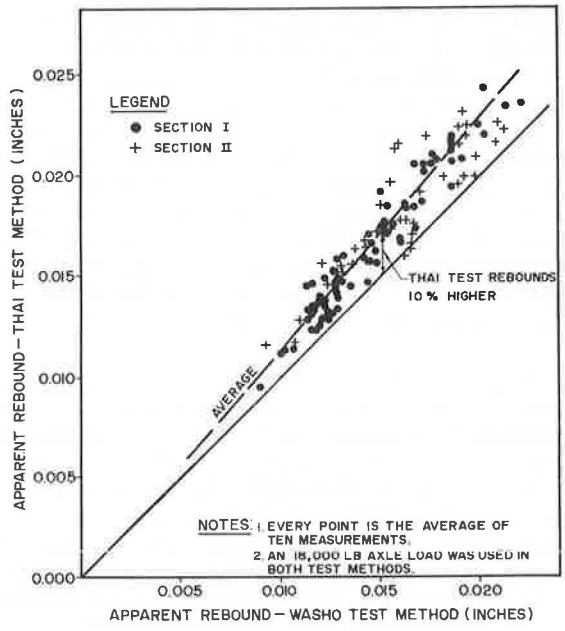
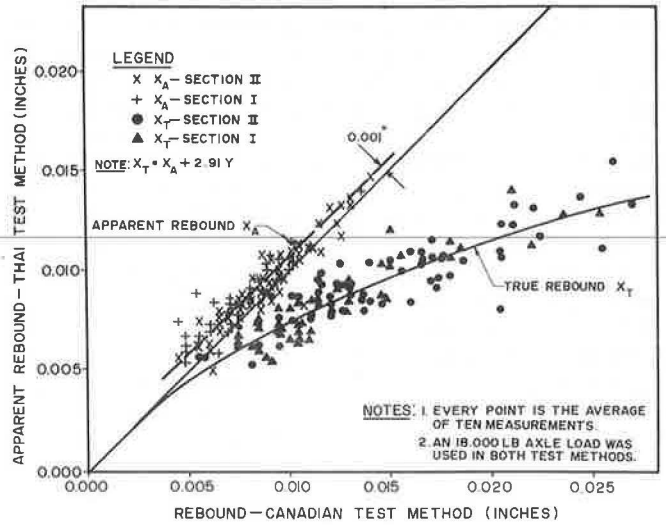


Figure 3. Correlation between Thai and Canadian test procedures.



Thon Buri-Pak Tho Highway because the residuals were found to be independent of leg movement. Similarly no correlation was found between residuals and pavement rebound. It was found, however, that the residuals increased from about 0.006 (0.15 mm) at pavement temperatures of 30 C to 0.010 in. (0.25 mm) at 45 C, indicating that shading effects could be important.

Temperature Correction Factor

The rebound deflection on asphalt concrete pavements has been found to depend on the pavement temperature, with the rebound generally increasing as the pavement temperature increases. The reason for this increased deflection is the weakening effect of higher temperatures on the asphalt concrete surfacing so that any temperature effect should depend on the proportion of the total pavement strength that the asphalt surfacing provides. The temperature correction factor that converts all deflection readings back to a standard temperature of 21 C in temperate climates (3) and 35 C in tropical climates (5) is therefore not a constant but depends on the pavement being tested.

The effect of temperature also seems to depend on the range of temperatures in which the tests are being carried out. In climates where pavement temperatures can vary from 0 to 30 C, the effect of temperature on deflections is significant because the asphalt is stiff at these low temperatures and contributes appreciably to the total pavement strength, especially in frost regions at the time of thaw.

In tropical countries, however, the pavement temperatures are often above 30 C as shown in Figure 5 for some typical temperature measurements throughout the day on the Thon Buri-Pak Tho Highway in Thailand. The effect of temperature is not so great at these higher temperatures, and Bulman (5) in Malaysia and Pawsey and Scala (11, Figure 5) in Australia show a much lower increase in deflection above 30 C than between 20 and 30 C. Carneiro (8) in Brazil found no effect of temperature on deflection when pavement temperatures exceeded 20 C, and Lister (1) also found a significant reduction in deflection above 30 C because of a plastic push-up between the wheels at these higher temperatures.

A trial was carried out on the Thon Buri-Pak Tho Highway to determine this temperature effect. Tests were carried out at ten locations at 330-ft (100-m) intervals every hour from 3 a.m. to 6 p.m. The Canadian test method was used, and the average apparent rebound and average true rebound for these 10 points were calculated for each hour. The pavement consisted of 2.5 in. (6.3 cm) of asphalt concrete, 8 in. (20 cm) of crushed rock, 6 in. (15 cm) of lateritic gravel, and 40 in. (102 cm) of sand overlying soft clay.

The results are shown in Figure 6, where it is seen that there is only a very small increase in apparent rebound of 0.001 in. (0.025 mm) as the pavement temperature increases from 30 to 40 C. There is, however, a small increase in the leg movement as pavement temperature increases, and the increase in the true rebound from 30 to 40 C is higher at 0.002 in. (0.05 mm).

The temperature correction equations used to correct all deflections to 35 C are therefore as follows (1 in. = 25.4 mm):

$$\text{Apparent rebound correction in inches} = 0.0001\ (35 - T_c) \qquad (2)$$

$$\text{True rebound correction in inches} = 0.0002\ (35 - T_c) \qquad (3)$$

where T_c = temperature in degrees Celsius.

The rebound correction coefficients of 0.0001 and 0.0002 are lower than those obtained in temperate or cold climates; e.g., CGRA (3) gives a coefficient of 0.0004 when the equation 0.0002 (70 - T_f) (temperature in degrees Fahrenheit) is converted to degrees Celsius; the graphs in Lister (1, Figure 2) show coefficients varying from

Figure 4. Correlation between WASHO and Canadian test procedures.

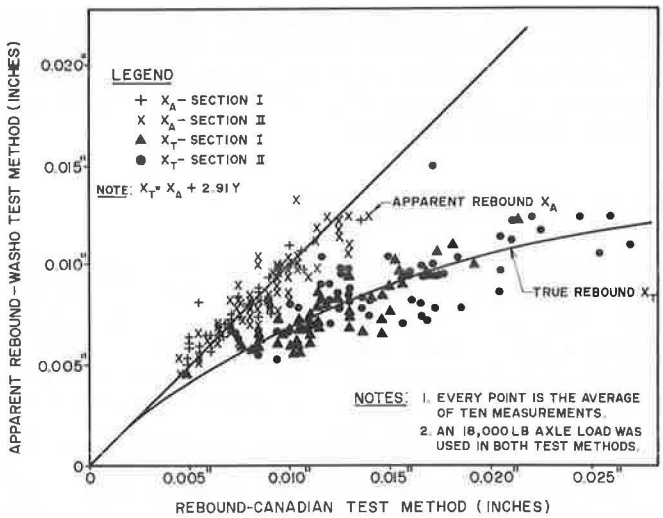


Figure 5. Typical daily variation in pavement temperature in Thailand.

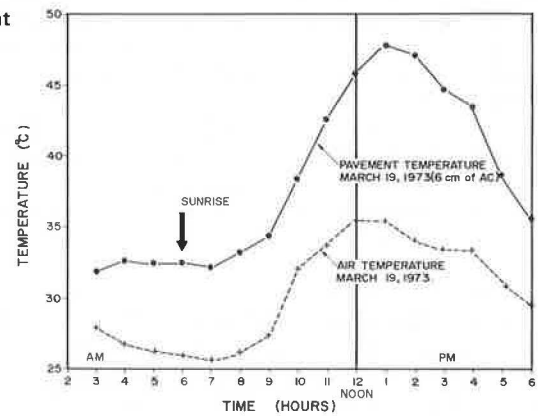
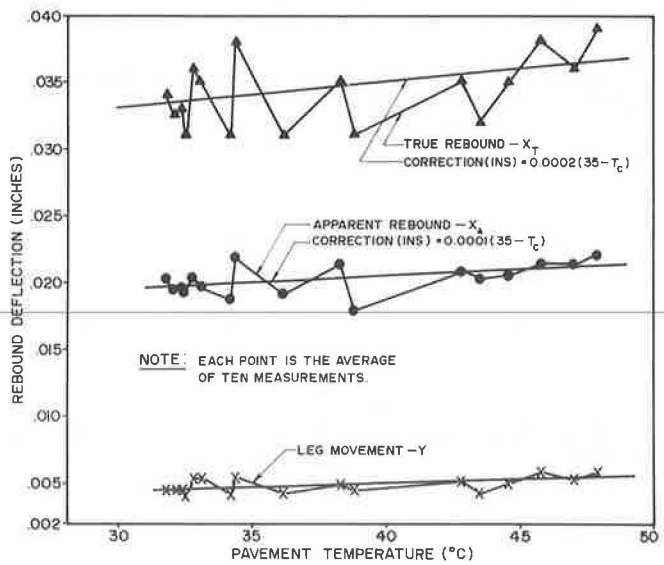


Figure 6. Results of temperature correction trial.



0.0003 to 0.0010 depending on the pavement type. The coefficients of 0.0001 and 0.0002 obtained are similar to other values obtained in tropical countries, e.g., the coefficient of 0.0002 derived from Figure 4 in Bulman (5) above 30 C and the zero values given elsewhere (8, 9).

These temperature trials confirm that the correction due to temperature is not as significant in tropical climates as it is in the temperate climates. The use in tropical climates of a standard temperature of 21 C and temperature correction equations derived from these colder temperate climates is dangerous because it can sometimes reduce measured deflection values by up to 50 percent and indicate that they are within acceptable limits. Corrections to a standard temperature of 35 C with the correction coefficient appropriate to tropical countries will give more realistic standardized deflection values.

Seasonal Variation in Deflection

Most authorities recommend taking deflection readings during the season when the pavement is weakest because most permanent deformation and consequent pavement distress occur in this period of the year (3, 4). In Thailand, the monsoon or wet season occurs in August and September, but water tables in this central deltaic region are highest in November when the combination of flood runoff plus high tides causes extensive flooding of from 8 to 30 in. (20 to 76 cm) in the lower deltaic area. A seasonal variation in deflection readings has been observed in this area (Figure 7) with maximum readings in November and minimum readings in May at the end of the dry season.

The seasonal deflection factor that is defined here as the ratio of the maximum deflection in November to that of the minimum deflection in May has been determined at several sites in the central plains deltaic area. Although this factor is shown as 1.32 in Figure 7, the average value for five sites with a bituminous pavement over a soft clay subgrade was 1.2. The seasonal deflection factor increased to about 1.5 for unsealed laterite pavements over soft clay subgrades where some reduction in the pavement strength as well as subgrade strength is likely in the wet season.

Curvature Measurements

A curvature meter that measured the middle ordinate of a 1-ft-long (0.3-m) arc was used to determine the maximum radius of curvature longitudinally between a set of twin tires at 12 different sites. All results are shown in Figure 8 in which the curvature measurements of intact pavements over firm clay subgrades generally agree with the relationship between deflection and radius of curvature given by Scala and Dickinson (12) and by the California Division of Highways (13). The radius of curvature for the pavements over the soft clay and swamp subgrades is however much higher for any given deflection. Alternatively, at any given radius of curvature the deflection is much higher, indicating that an appreciable percentage of the total deflection is occurring in the soft clay subgrade. These results do not agree with the conclusions of Dehlen (6), who states that the radius of curvature depends mainly on the elastic moduli of the upper pavement layers and little on the moduli at depth.

It is also evident from Figure 8 that failures were observed when the radius of curvature was less than 100 ft (30 m). This is similar to the critical figure of 125 ft (38 m) given by Dehlen (6) but smaller than the value of 200 ft (61 m) given by Zube and Bridges (14). If it can be shown that the critical design feature is the radius of curvature, then it is evident from Figure 8 that the allowable Benkelman beam deflection will increase as the subgrade soil becomes softer.

DESIGN DEFLECTION CRITERIA FOR PAVEMENT DESIGN FOR THON BURI-PAK THO HIGHWAY

The Thon Buri-Pak Tho Highway is a major arterial highway leading into Bangkok from the south of Thailand. The traffic was estimated at about 4,000 average daily traffic (ADT) (both lanes) in 1973 of which about 60 percent consisted of trucks and buses. A growth rate of 13 percent in the first year dropping linearly to 7 percent after 15 years was assumed in the traffic analysis, and this gave a cumulative number of standard axles of 7×10^6 in a 10-year design period based on the axle load factors in TRRL Road Note 29 (15).

The Canadian studies (3) indicated that, for roads carrying more than 1,000 ADT per lane (but only 10 percent trucks and buses), the final deflection ($X_T + 2\sigma$) in the critical spring period should be limited to between 0.030 and 0.050 in. (0.76 and 1.27 mm). The traffic conditions on the Thon Buri-Pak Tho Highway seem to be more severe because of the large number of commercial vehicles so that a maximum value ($X_T + 2\sigma$) of 0.040 in. (1.02 mm) should probably be chosen. In that σ for asphalt concrete pavements in most measurements on Thailand has averaged 0.005 in. (0.13 mm) this would require an average deflection X_T of 0.030 in. (0.76 mm).

The deflection criteria curves of the California Division of Highways (13) give (for a traffic index of 11 for heavy traffic) a required design deflection of 0.020 in. (0.5 mm) for 2.5 in. (6 cm) of asphalt concrete and 0.016 in. (0.4 mm) for 4.5 in. (11 cm) of asphalt concrete. Because we normally use an 18,000-lbf (80-kN) axle load for deflection testing compared with the 15,000-lbf (67-kN) standard axle load used in the California test procedure, the design deflection X_A for 4.5 in. (11 cm) of asphalt concrete must then be increased by this ratio to 0.019 in. (0.48 mm).

The deflection criterion curve given by Lister (1) and shown in Figure 9 gives a design deflection of 0.017 in. (0.43 mm) for a traffic loading of 7×10^6 standard axles. Converting from the standard British 14,000-lbf (62-kN) axle load to an 18,000-lbf (80-kN) axle load gives a design deflection X_A of 0.022 in. (0.58 mm).

The Thon Buri-Pak Tho Highway was designed for a 4.5-in. (11-cm) asphalt concrete surface layer, but only 2.5 in. (5 cm) will be placed during the construction period, and 2.0 in. (5 cm) will be placed several years after it has been opened to traffic. This is because extensive settlements of 4 to 20 in. (10 to 51 cm) are anticipated in the first few years because of the consolidation of the soft clay subgrade, and it was thought advisable to place this second 2.0-in. (5-cm) layer in conjunction with a leveling layer after the majority of the settlement had taken place. The Canadian, California, and British methods give design curves for the deflection reduction due to placing a 2.0-in. (5-cm) asphalt concrete overlay, and the required design deflection on top of the first 2.5-in. (6-cm) bituminous layer can be back figured as given in Table 1.

The interim true Canadian design deflection of 0.038 in. (0.97 mm) and the interim apparent British design deflections of 0.031 in. (0.79 mm) are probably comparable because the Canadian correction due to the front leg movement, which increases deflection values, is incorporated into all the empirical field studies in Canada used to determine these design deflections. The deflection required by the California method seems to be lower than that required by the other two methods, and as the method is based on more theoretical laboratory asphalt fatigue studies it will be disregarded from here on.

The only remaining point to be considered is whether these design deflections are applicable to the soft clay subgrades beneath the Thon Buri-Pak Tho Highway. Dormon (16) suggests two pavement design criteria: the limiting of (a) horizontal tensile strain in the bottom surface of the asphalt concrete and (b) vertical strain in the subgrade. It is evident that the first of these factors, the horizontal tensile strain, will be less with larger radii of curvature, and this should not therefore be a critical condition on these soils where radii of curvature are larger (Figure 8). If the design deflections determined from the field trials in Canada and England were due to failures arising from tensile fatigue, then they would be conservative for conditions in Thailand, and higher design deflections than those given in Table 1 could be used.

However, more recent work by the TRRL (17) has defined failure in terms of permanent pavement deformation, e.g., rutting depth criteria, and this indicates that the

Figure 7. Seasonal variation in deflection in central deltaic plains of Thailand.

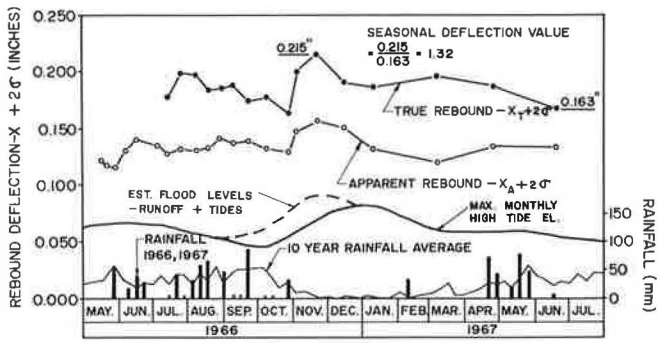


Figure 8. Radius of curvature measurements over various soil types.

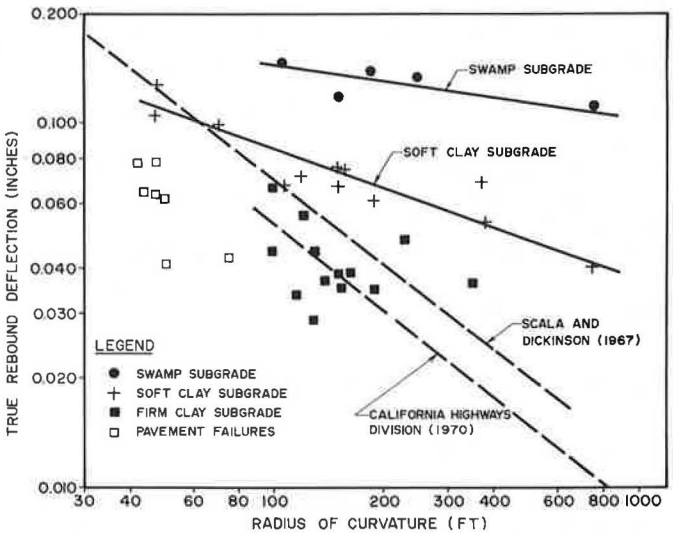


Figure 9. Design deflection criterion curves.

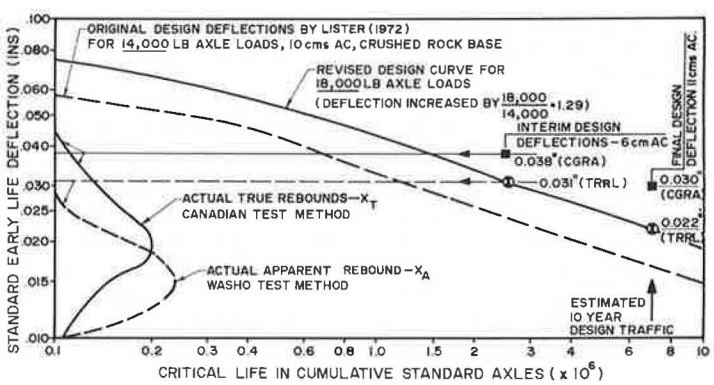


Table 1. Design deflection criteria for Thon Buri-Pak Tho Highway.

Design Method	Final Design Deflection for 4.5-In. Asphalt Concrete (in.)	Deflection Reduction From 2.0-In. Asphalt Concrete Overlay (in.)	Interim Design Deflection for 2.5-In. Asphalt Concrete (in.)
Canadian	0.030	0.008	0.038
British	0.022	0.009	0.031
California	0.019	0.007	0.026

Note: 1 in. = 25.4 mm.

limiting of vertical subgrade strain could be a more important criterion in pavement design. Scala and Dickinson (12) have shown that the reduction in Benkelman beam deflection from increased pavement thickness is due to a reduction in subgrade deflection in that the deflection in the crushed rock pavement layers was relatively constant irrespective of pavement thickness. It is thought therefore that the design deflection criteria established by the Canadian and British teams were derived to limit vertical subgrade strains and consequent permanent pavement deformations and are therefore applicable to the Thon Buri-Pak Tho Highway where strains in the soft clay subgrade must be limited to prevent rutting in the wheel tracks. An elastic stress analysis (18) for this highway also showed that this vertical subgrade stress was more critical than the horizontal tensile strain in the bituminous surfacing.

Confirmation of the design values in Table 1 has been obtained from one roadway in Thailand. An extensive analysis of cracked and uncracked sections on the Saraburi-Korat Highway (19) indicated a critical deflection value of 0.030 in. (0.76 mm). This value was the result, however, of using a standard pavement temperature of 21 C and the CGRA temperature correction equation (3). The critical deflection value at 35 C based on the temperature correction equation 0.0001 (35 - T_c) will be higher, and a revised critical deflection factor of 0.035 in. (0.89 mm) at 35 C can be obtained from their results.

The traffic index at the time of testing was calculated as 8.2 so that extrapolation to a traffic index of 11, done by drawing a line through a point with a deflection of 0.035 in. (0.89 mm) and a traffic index of 8.2, parallel to the California asphalt fatigue curves (13), gives a design deflection of 0.026 in. (0.66 mm). This value is intermediate between the British and Canadian values of 0.022 and 0.030 in. (0.56 and 0.76 mm) derived in Table 1 and confirms these design deflection criteria.

DEFLECTION MEASUREMENTS ON PAVEMENT LAYERS

Sand Embankment Fill

Deflection tests were carried out at various levels during the construction of the sand embankment. The embankment was built by pushing a 30-in. (75-cm) working blanket of uniform fine sand forward over the soft soils and then building it up in 8-in. (20-cm) lifts to a final thickness of 3 to 5 ft (0.9 to 1.5 m). It was found that the embankment was very bouncy immediately after a new lift was placed and that it set up in about 2 weeks after the pore pressures formed in the surface layers from the new lift had dissipated.

At a sand embankment height of 30 in. (75 cm), the reduction in deflection per 8-in. (20-cm) lift height was found to be 0.008 to 0.01 in. (0.2 to 0.25 mm), and the percentage of reduction in deflection for an 8-in. (20-cm) lift varied from 10 to 15 percent. The California overlay design curve (13) gives an equivalent granular thickness of 0.8 in. (2 cm) or an equivalency factor of only 0.10 for this percentage reduction; however, the Canadian overlay design method (3) gives an equivalency factor of 0.10 to 0.15.

This equivalency factor for sand seems low and is much lower than the value of 0.5 to 0.75 generally assumed for this material. This result indicates either that the sand is a poor material for reducing deflections or that this approach is not valid where the depth over the soft clay subgrade is small.

Lateritic Gravel Subbase

Deflection tests were also carried out on top of the lateritic gravel subbase and consisted of 6.5-in. (16.5-cm) thickness of 2-in. (5-cm) maximum size lateritic gravel blended with 10 to 20 percent sand to reduce its plasticity and improve its gradation characteristics. The average deflection reduction realized was only equivalent to using 2.5 in. (6.5 cm) of crushed rock and gave an equivalency factor of 2.5/6.5 = 0.40, which

Table 2. Cost effectiveness of pavement layers.

Item	Cost (dollars/m³)	Material Cost Relative to Base Course	Deflection Reduction Relative to Base Course	Ratio of Deflection Reduction to Cost[a]
Asphalt concrete	24.50	3.0	2.0	0.67
Crushed-rock base course	8.25	1.0	1.0	1.00
Lateritic gravel subbase	5.25	0.65	0.40	0.61
Sand embankment	3.00	0.36	0.10 to 0.15	0.40

Note: $1/m³ = $35/ft³.

[a]4:3.

is again quite low in that the lateritic subbase material looked like a good material when placed and compacted.

Crushed Rock Base

The crushed rock used for the 8-in. (20-cm) nominal base course thickness was constructed in two layers of 4-in. (10-cm) thickness. A comparison between the actual deflection reduction and the overlay design curves of Canada and California indicated that the crushed rock base was behaving normally, with an equivalency factor of 1.0. The CGRA overlay design curve seemed to fit the results better.

Asphalt Concrete Surfacing

The asphalt concrete surfacing was actually 2.0 to 2.5 in. (5 to 6.2 cm) thick. The deflection reduction immediately after the bituminous surfacing was placed was compared with the overlay design curves, and it was found that the surfacing was behaving normally and that an equivalency factor of 2.0 could be assumed. This is in contrast to the findings of Scala and Dickinson (12), who found a lower factor than 2.0 in hotter climates.

The deflections on the final asphalt concrete surface were found to decrease after traffic, as has been noted previously by Lister (1).

Comparison With Design Deflection Criteria

The distribution of the actual rebounds obtained on the final asphalt concrete surface is plotted in conjunction with the TRRL design curve in Figure 9. It is seen that the actual mean deflections of 0.02 and 0.015 in. (0.5 and 0.38 mm) for the Canadian and WASHO test methods respectively are well below the corresponding design deflections of 0.038 in. (0.97 mm) and 0.031 in. (0.79 mm) respectively given in Table 1.

The spread of the Canadian deflections is much wider than the WASHO deflections and is a reflection of the magnifying effect of the leg movements over the softer soil types. These results confirm that the pavement as constructed was satisfactory because the final deflection results obtained are well below the design limits.

Cost Effectiveness of Various Pavement Layers

The average contract unit rates of the materials used in the construction of the Thon Buri-Pak Tho Highway, the deflection reduction figures determined for various pavement layers, and the ratio of the deflection reduction performance to the cost of the material are given in Table 2.

Table 2 indicates that the base course is the most economical material to use to decrease deflections on these soft clay subgrades; the next most cost-effective materials

are asphalt concrete, lateritic gravel, and then sand. Table 2 provided the economic basis for increasing the base course thickness from 6 to 8 in. (15 to 20 cm) and for decreasing the subbase thickness from 12 to 6 in. (30 to 15 cm). This resulted in the same deflection values being obtained while savings of $700,000 were realized.

The sand embankment fill performed badly in relation to its cost. However, the main purpose of the sand fill is not to reduce deflection values but to build up the height of the roadway embankment until it is clear of the floodwater level in this deltaic area and to exert as little pressure on the soft clay subgrade as possible. The sand used was a fine uniform sand with a low density of 100 lb/ft^3 (1602 kg/m^3), which is over 30 percent lighter than the crushed rock. The sand embankment material therefore fulfilled its role and, therefore, the upper pavement layers were to reduce deflection values to acceptable values.

The economic advantages of using the maximum amount of crushed rock base course material are evident from Table 2. Unfortunately, current practice in Thailand has been to use minimum base course thicknesses of 6 in. (15 cm) for most main highways. Such thicknesses are low when compared to the values given in TRRL Road Note 29 (15) and when the economic advantages obtained in the above deflection studies are considered. Base course thicknesses of 8 to 10 in. (20 to 25 cm) for main highways over this central deltaic area in Thailand are therefore recommended for future highways.

DEFLECTION MEASUREMENTS AFTER CONSTRUCTION

Deflection measurements were taken 7 months after the highway was opened to traffic, and the results indicated a rapid increase in deflection during this period. The average apparent deflection X_A increased from about 0.015 to 0.020 in. (0.38 to 0.50 mm) even though the number of cumulative standard axles was only 0.1×10^6 in this period. The deflections are still, however, below the design deflections. This effect is currently being considered in more detail, and two possible reasons are being investigated:

1. The settlement may be causing tensile strains in the crushed rock pavement layer, which is weakening the pavement layers; and
2. The live load repetitions may be weakening the overconsolidated surface layers of the soft clay subgrade [the softening effect of transient loadings on overconsolidated clays has been demonstrated by Bishop and Henkel (20)].

CONCLUSIONS

Extensive Benkelman beam testing was carried out during the design and construction of the Thon Buri-Pak Tho Highway. The testing was more thorough and complex than normal because of the unknown effect of the soft clay subgrade beneath the highway and because certain testing techniques have had to be investigated during the course of this research.

The testing investigations have shown that a seasonal deflection effect occurs on the soft deltaic soils in the central plains area of Thailand and that the temperature correction factor is much less in tropical countries than in temperate climates. The Canadian test procedure was found to give a more accurate rebound figure than the WASHO test procedure because it could take account of the large movements of the front leg of the Benkelman beam on this soft soil subgrade.

Benkelman beam tests were carried out during construction, and the equivalency factors determined indicated that crushed rock was the most effective material in reducing deflections because its cost was lower than that of asphalt concrete, lateritic gravel, and sand. These investigations caused the pavement to be redesigned for a greater thickness of crushed rock base and a smaller thickness of lateritic gravel subbase. This resulted in substantial construction cost savings of $700,000.

The average actual deflection readings on the interim 2.5-in. (6.4-cm) pavement thickness were 0.020 and 0.015 in. (0.5 and 0.38 mm) for the Canadian and WASHO test

methods respectively, but the corresponding design deflections at this interim stage were 0.038 and 0.028 in. (0.97 and 0.71 mm). The actual deflections averaged therefore about half of the design deflections, and practically all readings fell below these design limits. There has been a rapid increase in deflection after only 7 months of traffic and 0.1×10^6 cumulative standard axles to give an average deflection of about 0.020 in. (0.5 mm) by the WASHO test method. The deflections are still however below the design deflection, and the final pavement as constructed was therefore considered to be satisfactory.

ACKNOWLEDGMENT

I would like to thank the Thailand Department of Highways for permission to publish this paper. I would also like to thank Nibon Rananand and Vicharn Poopath of the Materials and Research Division of the department for their assistance in carrying out the many tests described in this paper.

REFERENCES

1. N. W. Lister. Deflection Criteria for Flexible Pavements and the Design of Overlays. Proc., 3rd International Conference on Structural Design of Asphalt Pavements, London, 1972.
2. E. Zube and R. Forsyth. Flexible Pavement Maintenance Requirements as Determined by Deflection Measurements. Highway Research Record 129, 1966, pp. 60-76.
3. Pavement Design and Evaluation Committee. A Guide to the Structural Design of Flexible and Rigid Pavements in Canada. Canadian Good Roads Association, 1965.
4. N. W. Lister. Deflection Criteria for Flexible Pavements. Road Research Laboratory, U.K. Ministry of Transport, Rept. LR 375, 1971.
5. J. N. Bulman. Strengthening of Flexible Roads in the Tropics. U.K. Transport and Road Research Laboratory, TRRL Rept. LR 444, 1972.
6. G. L. Dehlen. Flexure of a Road Surfacing, Its Relation to Fatigue Cracking and Factors Determining Its Severity. HRB Bulletin 321, 1962, p. 26.
7. The WASHO Road Test, Part 1—Design, Construction, and Testing Procedures. HRB Special Rept. 18, 1954.
8. F. B. L. Carneiro. Benkelman Beam—Auxiliary Instrument of the Maintenance Engineer. Highway Research Record 129, 1966, pp. 28-59.
9. Deflection Beam Measurements on Roads in Kenya. Road Research Laboratory, U.K. Ministry of Transport, Unpublished Note LN/508/KR, 1964.
10. A. J. Scala. Pavement Performance Using Residuals in the Deflection Test. Proc., 3rd International Conference on Structural Design of Asphalt Pavements, London, 1972.
11. M. J. Pawsey and A. J. Scala. Some Effects of Temperature of Bituminous Layers on Deflection. Proc., 6th Conference of Australian Road Research Board, Canberra, 1972.
12. A. J. Scala and E. J. Dickinson. The Use of Asphalt Pavement Structures in the Australian Environment. Proc., 2nd International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1967, pp. 963-973.
13. Materials Manual. California Division of Highways, 1970.
14. E. Zube and R. Bridges. The Use of Pavement Deflections in Asphalt Pavement Overlay Design. Proc., 1st International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1962.
15. A Guide to the Structural Design of Pavements for New Roads. Road Research Laboratory, U.K. Ministry of Transport, 3rd Ed., Road Note 29, 1970.
16. G. M. Dormon. The Extension to Practice of a Fundamental Procedure for the Design of Flexible Pavement. Proc., 1st International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1962, p. 785.

17. D. Croney. Failure Criteria for Flexible Pavements. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, 1972, pp. 608-612.
18. W. Heukelom and A. J. G. Klomp. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proc., 1st International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1962, p. 785.
19. Pavement Deflection Study of Highway Route 2 Between Saraburi and Korat. De Leuw Cather International, 1967.
20. A. W. Bishop and D. Henkel. Pore Pressure Changes During Shear in Two Undisturbed Clays. Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1953, p. 94.