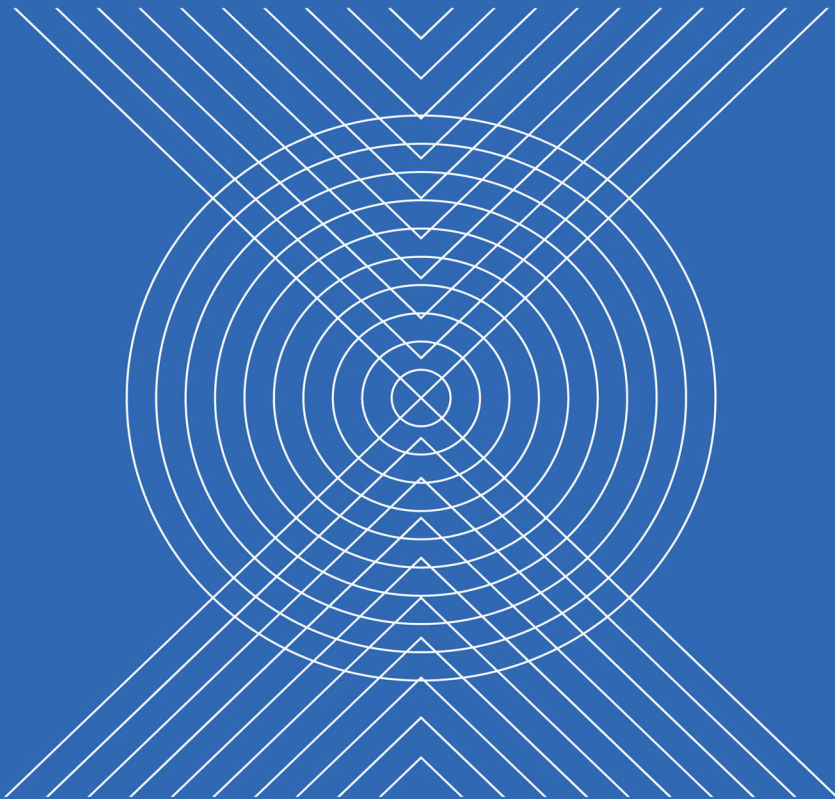


Civil, Architecture and Environmental Engineering

Editors: Jimmy C.M. Kao & Wen-Pei Sung



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Preface

The 2016 International Conference on Civil, Architecture and Environmental Engineering (ICCAE 2016) was held on November 4-6, 2016 in Taipei, Taiwan, organized by China University of Technology and Taiwan Society of Construction Engineers, aimed to gather professors, researchers, scholars and industrial pioneers from all over the world. ICCAE 2016 is the premier forum for the presentation and exchange of experiences, new advances and research results in the field of theoretical and industrial experience. The conference contained contributions promoting the exchange of ideas and rational discourse between educators and researchers from all over the world.

ICCAE 2016 is expected to be one of the most comprehensive Conferences focused on civil, architecture and environmental engineering. The conference promotes international academic cooperation and communication, and exchanging research ideas.

We would like to thank the conference chairs, organization staff, and authors for their hard work. By gathering together so many leading experts from the civil, architecture and environmental engineering fields, we believe this conference has been a very enriching experience for all participants. We hope all have had a productive conference and enjoyable time in Taipei!

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Modeling two-dimensional rubble mound breakwater using dolos at armor layer and geotube at the core layer

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ABSTRACT: Coastal protection structure is used to protect the coast against erosion that is usually determined by the availability of materials at or near the job site, the sea conditions, water depth, and the availability of equipment for the implementation of the work. The purpose of this study is to conduct experimental research in the laboratory to obtain the rubble-mound breakwater design optimum and tools before it is applied directly in the field to reduce the risk of failure of construction. This study is conducted for laboratory testing of a two-dimensional rubble-mound breakwaters model with dolos at armor layer and geotube at core layer using regular wave and three variations of slope in front of the structure facing seaward, that is, 1:1.5, 1:2, and 1:2.5. The water level varies for the conditions of non-over-topping, over-topping, and submerged. The two-dimensional rubble-mound breakwaters model with the slope in front of the structure facing seaward 1:2.5 shows the most stable rubble-mound breakwater model.

1 INTRODUCTION

Indonesia as an archipelago country does not escape the impact of global warming due to the sea level rise. The impact of sea level rise results in the coastline change due to erosion and/or abrasion. In addition to global warming, forces of nature such as waves, currents, tides, and construction errors caused major damage to the existing breakwater. In general, construction fault lies in the tilt angle of the building, a thick layer, and heavy rock, in addition to other factors. This can lead to the collapse of the breakwater.

Breakwater is a structure on the coast to protect the coast against erosion. The type of breakwater structure used is generally determined by the availability of materials in or near the work site, the sea conditions, water depth, and the availability of equipment for the implementation of the work. Rubble-mound breakwaters use structural voids to dissipate the wave energy. Rock or concrete armor units on the outside of the structure absorb most of the energy, while gravels or sands prevent the wave energy's continuing through the breakwater core.

Construction of a breakwater on the field will take considerable time and cost. A laboratory test using a physical model is proved to be effective in avoiding such costs before constructing breakwater in the field. In this study, we will discuss the optimization of two-dimensional model of the rubble-mound breakwater using dolos at the armor layer

and geotube at the core layer, because it has a high permeability that balances the water pressure. Wave run-up will be included in determining the peak elevation breakwater.

2 COASTAL PROTECTION STRUCTURE

Coastal structures are intended to protect shoreline or navigation channels from the effects of waves and other hydrodynamic force (Fith, et al., 2014). Design of coastal structures must consider a range of wave heights and periods combined with water level variations. The most common types of coastal structures are breakwaters. Breakwater is a structure constructed on coasts as part of coastal protection from the effects of both weather and longshore drift. Breakwater reduces the intensity of wave action in inshore waters and thereby reduces coastal erosion. Breakwaters may also be small structures designed to protect a gently sloping beach and placed at a distance of 1–300 feet offshore in relatively shallow water. Breakwaters can be constructed with one end linked to the shore, in which case they are usually classified as sea walls; otherwise, they are positioned offshore from as little as 100 m up to 300–600 m from the original shoreline. The types of breakwaters are rubble-mound structures, impermeable sloping structures, vertical wall structures, composite

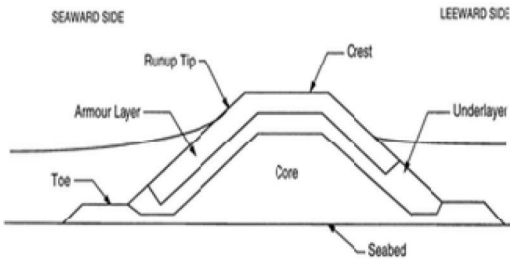


Figure 1. Cross section of a typical rubble-mound breakwater.

structures, floating structures, and pneumatic and hydraulic breakwaters (Hughes, 1993).

Rubble-mound structure is used extensively for breakwaters, jetties, revetments, seawalls, and wave absorber. Rubble-mound structures have a core of quarry-run stones, sand, slag, or other suitable materials protected from waves action by one or more stone under layers and a core layers of relatively large, selected quarry stones or specially shaped concrete armor units (Hudson, 1959). Rubble-mound breakwaters use structural voids to dissipate the wave energy. Rock or concrete armor units on the outside of the structure absorb most of the energy, while gravels or sands prevent the wave energy's continuing through the breakwater core. The integrity of a rubble-mound structure is primarily a function of the stability of the individual armor units that form the seaward face of the structure. In shallow water, revetment breakwaters are usually relatively inexpensive. As water depth increases, the material requirements and, hence, costs increase significantly. Figure 1 shows a typical cross section of a rubble-mound breakwater.

3 STABILITY OF RUBBLE-MOUND BREAKWATER

An important aspect in the design of a rubble mound is its stability to wave attack. Three aspects of the effect of waves on rubble-mound breakwater are wave run-up, overtopping, and transmission (Figure 2). Run-up is defined as the vertical height above still water level to which waves incident upon a structure can be expected to travel up the face of the structure. Wave run-up is important in defining both the amount of wave energy transmitted over and through permeable rubble-mounds and also the quantity of water that may be expected to overtop the structure (Hur, Lee, and Cho, 2012).

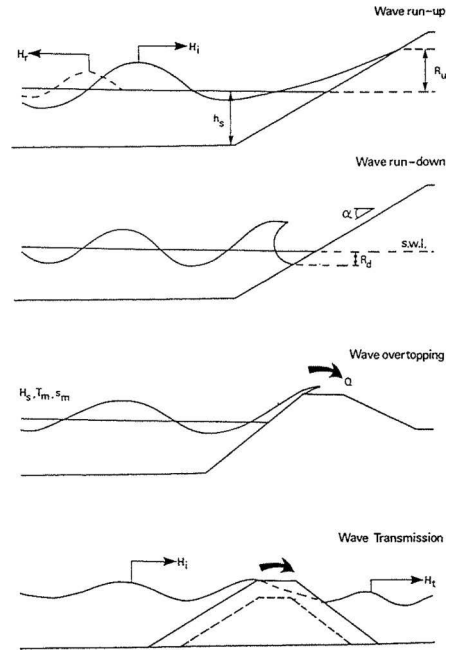


Figure 2. Waves on rubble-mound breakwater.

4 EXPERIMENT SETUP AND PROCEDURE

The experimental test was conducted on the wave flume at Laboratory Balai Pantai, Ministry of Public Works in Buleleng, Bali, Indonesia. The wave flume is 40 m long, 0.6 m wide and 1.2 m high. It is equipped with a piston-type wave generator that produces regular wave. Figure 3 shows three wave probes that are installed in the wave flume.

Simulation of the wave generated by the wave maker uses regular wave model. The model of rubble-mound breakwater is constructed with the seaward face at slopes 1:1.5, 1:2, and 1:2.5 using an armor layer composed of individual units of weight W and density ρ . The armor units use dolos with the average weight of 265 g, relative density of 2.209 g/cm³, and K_d of 8. The dolos is randomly placed at the armor layer (CIRIA, 2007). The core layer is set to be stable by using geotube. Three variations of slopes facing the seaward side of the breakwater model are applied: 1:1.5, 1:2, and 1:2.5. The wave period is 2 s. The wave height for different slopes of breakwater models can be seen in Table 1.

The water level tests are conducted at four different water levels: 38 cm (mean low water level), 54 cm (mean high water level), 63 cm (crest elevation), and 70 cm (submerged). The structure is assumed to be founded on an impermeable bed, and the duration of exposure to waves is not considered.

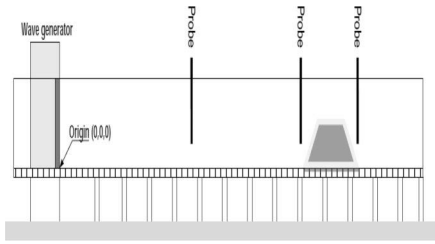


Figure 3. Side view of the flume.

Table 1. Wave heights at different slope of breakwater model.

Slope	Wave height (cm)
1:1.5	12.42
1:2	13.66
1:2.5	14.72

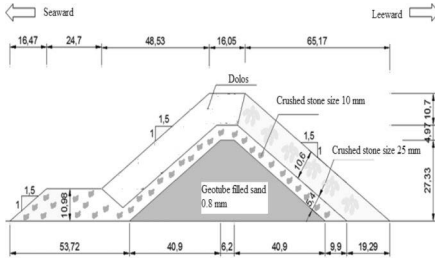


Figure 4. Side view of rubble-mound breakwater model with slope of 1:1.5.

The rubble-mound breakwater model is designed on the basis of the stability of rubble-mound structure using the Hudson formula (Hudson, 1959). The similarity of the rubble-mound breakwater model is 1:10. Figure 4 presents the side view of the rubble-mound breakwater model with the slope of 1:1.5.

5 RESULT AND DISCUSSION

Wave direction, either normally incident or oblique, refers to the direction of wave travel with respect to the breakwater axis. Tables 2–4 present the results of wave height measured using zero upcrossing statistic in front of rubble-mound structure model with slopes 1:1.5, 1:2, and 1:2.5 facing the seaward side, respectively. The highest wave height occurred at crest elevation.

The significant wave height, runup, and rundown at 38 cm of water elevation can be seen in Table 5. It can be concluded that $H_{1/3}$ and $R_u - R_d$ at the mean low water level show the lowest values at the

Table 2. Wave heights based on zero upcrossing statistics in front of rubble-mound structure with slope 1:1.5 facing the seaward side.

Water level (cm)	H_{\max} (cm)	$H_{1/3}$ (cm)	$H_{1/10}$ (cm)	H_{average} (cm)
70	18.0	15.5	16.7	14.4
63	27.0	25.8	26.3	23.9
54	16.2	15.4	15.9	14.1
38	15.5	13.7	14.5	11.5

Table 3. Wave heights based on zero upcrossing statistics in front of rubble-mound structure with slope 1:2 facing the seaward side.

Water level (cm)	H_{\max} (cm)	$H_{1/3}$ (cm)	$H_{1/10}$ (cm)	H_{average} (cm)
70	18.5	16.2	16.7	13.4
63	19.9	18.5	19.1	16.7
54	14.3	14.0	14.1	13.1
38	15.9	14.5	15.0	13.0

Table 4. Wave heights based on zero upcrossing statistics in front of rubble-mound structure with slope 1:2.5 facing the seaward side.

Water level (cm)	H_{\max} (cm)	$H_{1/3}$ (cm)	$H_{1/10}$ (cm)	H_{average} (cm)
70	17.9	15.7	16.3	14.9
63	18.1	17.3	17.7	16.0
54	17.6	16.7	17.1	15.4
38	13.8	12.6	13.1	11.3

Table 5. Significant wave height, runup, and rundown at 38 cm of water elevation.

Format of slope	$H_{1/3}$ (cm)	Runup, R_u (cm)	Rundown, R_d (cm)	$R_u - R_d$ (cm)
1:1.5	13.7	32.0	8.5	23.5
1:2	14.5	28.0	8.0	20.0
1:2.5	12.6	20.0	6.0	14.0

slope 1:2.5 of the rubble-mound breakwater model. It means that the highest energy of wave absorption occurred at the slope of 1:2.5 facing the seaward side of the rubble-mound breakwater model.

When the water level is 54 cm or at the mean high water level, only rundown occurred. The rundown for each different slope of the rubble-mound breakwater model is shown in Table 6. The wave absorber gives the highest value for the slope of 1:2.5 for the rubble-mound breakwater model.

The elevation of the crest should be minimum at which overtopping occurs. This should be based on maximum wave runup (Palmer and Christian,

Table 6. Rundown at the mean high water level.

Format of slope	Rundown, R_d (cm)
1:1.5	14.5
1:2	17.0
1:2.5	22.0

Table 7. Wave transmission coefficient at the water level 70 cm.

Slope	H_t (cm)	H_i (cm)	K_t
1:1.5	13.1	15.5	0.85
1:2.0	9.0	16.2	0.56
1:2.5	8.3	15.7	0.53

1998). Unfortunately, the overtopping at 63 cm of water elevation cannot be recorded due to the instrument's problem during the experiment.

The wave transmission coefficient (K_t) is defined as the ratio of the transmitted wave height (H_t) at the leeside to the incident wave height (H_i) at the breakwater seaward, as follows:

$$K_t = \frac{H_t}{H_i} \quad (1)$$

The value of K_t indicates the effectiveness of a rubble-mound breakwater at submerged condition to attenuate waves (Yuliasuti and Hashim, 2011, Zhang and Li, 2014). The value of K_t varies between 0 and 1. A value of zero implies that there is no transmission, whereas 1 means no reduction in wave height (there is no barrier in front of the wave). The result of wave transmission coefficient is presented in Table 7 at the water level of 70 cm (submerged).

On the basis of the result of wave transmission coefficient, slope 1:2.5 of the rubble-mound breakwater model can reduce the wave height more significantly than the slopes 1:1.5 and 1:2 of the breakwater model.

6 CONCLUSIONS

The function of rubble-mound breakwater is to protect the coastal area. Because the rubble-mound breakwater needs to be stable, the two-dimensional physical model of rubble-mound breakwater is conducted using three different slopes, 1:1.5, 1:2, and 1:2.5, facing the seaward side. The water levels are applied at the mean low water level, the mean high water level, the crest elevation, and submerged using the flume with regular wave. The rubble-mound breakwater model is designed on the basis of stability of this breakwater using the Hudson formula. The armor units of the rubble-mound breakwater model are dolos placed randomly facing the

seaward side. The core is set stable using geotube. The wave absorber, transmission wave, and displacement of armor units give the best result for the slope of 1:2.5 of the rubble-mound structure model.

The results of two-dimensional physical testing for this rubble-mound breakwater model will certainly be helpful in making the optimum slope of rubble-mound breakwater facing the seaward side in coastal areas of Indonesia with erosion and/or abrasion condition. The results of the two-dimensional rubble-mound breakwater model will be very useful if it can proceed to the next stage of the three-dimensional physical test by using regular and irregular waves. In addition, test results of physical model of this structure can also improve the existing rubble-mound breakwater, especially damaged rubble-mound breakwater.

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Shear stress calculation of rubber asphalt overlay and stress-absorbing layer

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ABSTRACT: By using BISAR mechanical calculation software to calculate and analyze the rubber asphalt compound pavement shear stress of different interlayer adhesive states and under the condition of stress-absorbing layer thickness, we know that the adhesive states of stress-absorbing layer, rubber asphalt surface layer, and underlying cement pavement have a direct impact on the distribution of shear stress of asphalt overlay and stress-absorbing layer, and the maximum shear stress of asphalt layer bottom will first decrease and then increase with the increase of friction parameters. Therefore, the bottom layer should have good adhesion to prevent the premature entry of shear stress growth period.

1 PREFACE

The stress-absorbing layer is arranged on the general cement concrete deck and asphalt surface layer, which can effectively disperse the stress concentration of cement concrete joints or cracks under load effect in order to delay or prevent the appearance of reflective crack of asphalt pavement. The commonly used stress-absorbing layer types mainly include the layer distribution method stress absorbing layer (such as SAMI), modified asphalt mixture stress absorbing layer, the compound stress absorbing interlayer, geosynthetics, and synchronous macadam transition layer, among which the first three have better impermeable, crack resisting, and bonding properties, whereas the effect of the latter two in the practical application is not significant. The cost of modified asphalt stress absorbing layer and compound stress absorption interlayer engineering is high, and the difficulty of construction is relatively high. By contrast, the layer distribution method stress absorbing layer is relatively more economical, reasonable, and convenient for construction. Rubber powder-modified asphalt with high elasticity, toughness, and high deformation is

more applicable to the layer distribution method stress absorbing layer. In addition, the rubber powder-modified asphalt stress absorbing layer is also an effective measure of waste tires.

The layer distribution method for rubber powder-modified asphalt stress absorption layer is widely used in compound pavement engineering; however, some technical problems still remain to be solved, including setting rubber powder-modified asphalt binder stress-absorbing layer asphalt overlay design theory and method of structure, the rubber asphalt binding material to improve the technical performance, and the construction technology and other aspects. In light of this, the above rubber asphalt stress-absorbing layer shear characteristics are studied in this paper, in order to further improve the technology of rubber powder-modified stress absorbing layers.

2 ANALYSIS OF THE INTERLAMELLAR SHEAR STRESS

The combination form of general rubber asphalt compound pavement structure is shown in Fig. 1.

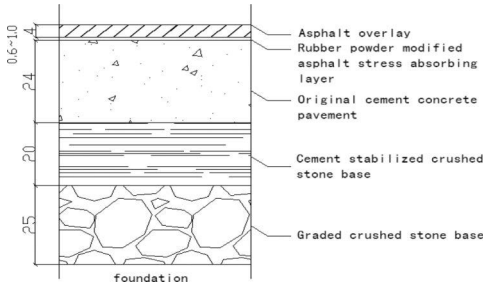


Figure 1. Pavement structure combination.

Table 1. Pavement structure parameters.

Structure layer	Thickness (cm)	Modulus (MPa)	Poisson's ratio
Asphalt overlay	4.0	1200	0.35
Absorbing layer	0.6-1.0	400	0.35
Cement concrete	24.0	30000	0.15
Cement-stabilized stone base	20.0	1500	0.20
Graded stone	25.0	350	0.35
Soil matrix	-	100	0.40

In addition to the rubber powder-modified asphalt stress-absorbing layer structure of each layer of compressive resilient modulus and Poisson's ratio, we refer to "asphalt pavement design specification" -JTG D50-2006 value. However, the rubber powder-modified asphalt stress-absorbing layer refers to the modified asphalt stress-absorbing layer parameters. The values of the pavement material parameters for each layer are shown in Table 1.

Using BISAR pavement structure in Table 1 in the interlayer under BZZ-100 standard axle load conditions, shear stress is calculated to analyze the thickness of the shear stress distribution, contact conditions, the interlayer shear stress, and the effects of stress-absorbing layer thickness on shear stress.

2.1 Distribution of shear stress in the lower layers of different adhesive states

In this analysis, the stress-absorbing layer and the upper and lower positions under different conditions of friction parameters ∂ each point along the driving direction (0,1a,2a,3a) at the XX and YY normal stress component analysis asphalt surface and the bottom of the maximum stress absorbing layer shear stress distribution. The results are shown in Figures 2-6.

As can be seen from Figures 2-6, the maximum shear stress distribution of the stress-absorbing compound pavement overlay and the stress absorbing underlying layer can be changed with different adhesive statuses, and the main changes are as follows:

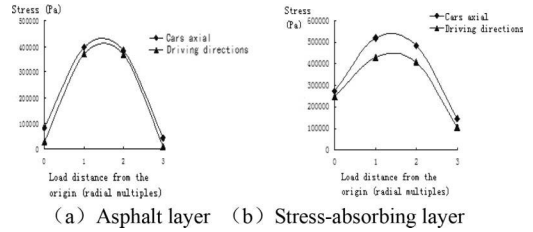


Figure 2. Shear stress at the underlying layer under the condition of complete continuity between two layers.

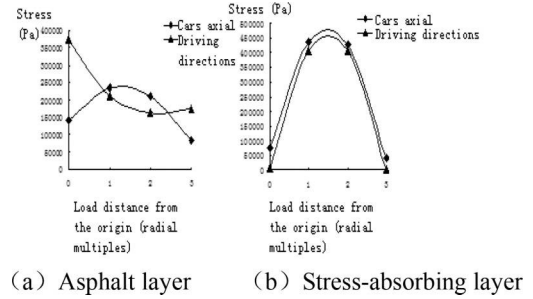


Figure 3. Shear stress at the underlying layer when the upper layer is 0.5.

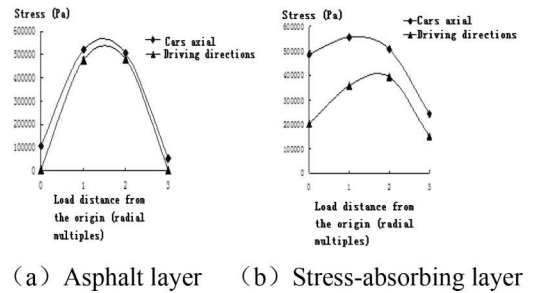


Figure 4. Shear stress at the underlying layer when the upper layer is completely sliding.

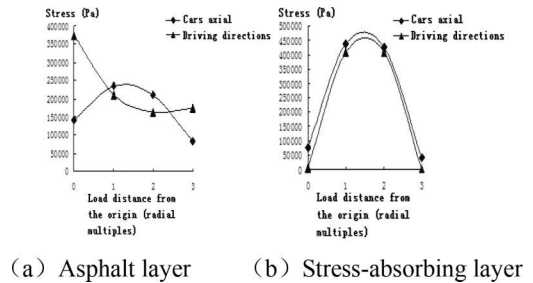
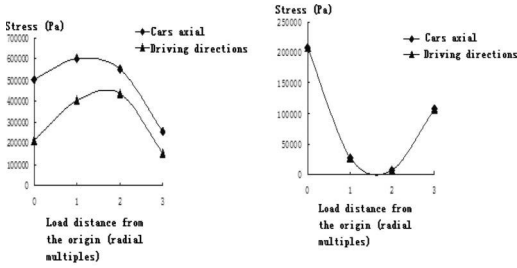


Figure 5. Stress absorbing layer friction parameter is 0.5.



(a) Asphalt layer (b) Stress-absorbing layer

Figure 6. Shear stress at the underlying layer when the absorbing layer is completely sliding.

Table 2. Shear compliance under different friction parameters.

Adhesive layers	Friction parameters	Modulus (MPa)	Poisson's ratio	AK (N/m ²)
Asphalt overlay and stress-absorbing layer	0	1200	0.35	0
	0.25			3.99E-11
	0.5			1.20E-10
	0.75			3.59E-10
Stress-absorbing layer with the original concrete pavement	0	400	0.35	1.19E-08
	0.25			1.20E-10
	0.5			3.59E-10
	0.75			1.08E-09
	0.99			3.56E-08

1. When every level of compound pavement is completely in continuous state, asphalt layer and stress-absorbing layer stress was basically consistent along the direction of travel distribution, and auto axial and driving direction stress are all parabolas, and the axial stress is always greater than the direction of traffic stress.
2. When an adhesive status between the asphalt surface layer and the stress-absorbing layer changes, the distribution of stress in asphalt overlay takes a significant change along the directions of traffic. When the friction parameter is in the case of 0.5, the axial stress and stress variation of the direction of travel are exactly opposite, and the maximum shear stress is travel direction stress in load center, which proves that the interlayer adhesive status affects the stress distribution of the asphalt surface layer and the maximum shear stress direction. At the same time, under three friction parameter conditions, absorbing layer stress changing rules along the direction of traffic are not obvious, but there still exists possibility of the presence of the influence of different friction parameters on stress variation.

3. When the stress-absorbing layer and the original cement coagulation pavement adhesive status change, the variation of asphalt overlay and stress-absorbing layer of stress were significantly changed. When the friction parameter is 0.5, that is to say, when the stress-absorbing layer and the cement concrete road surface are not fully bonded, the stress distribution is basically equal to the effect generated the asphalt layer—the changing adhesive status of the stress-absorbing layer. Changes in stress distribution stress-absorbing layer occurs when incomplete bonding is not large; however, when the layers completely coincide, the driving direction and the distribution of the axial stress are exactly opposite with fully bonded case, namely the maximum shear stress is seen as a parabola, which decreases first and then increases.
4. From the distribution of stress in different layers and the variation of bonded conditions, it can be seen that the interlayer shear stress distribution varies, and this effect will change with the variation of friction parameters.

2.2 Variation of maximum shear stress between the layers with different bond states

BISAR is used to make mechanical analysis for stress-absorbing layer and the upper and lower positions, respectively, under different conditions of friction parameters. The friction parameters were 0, 0.25, 0.5, 0.75, and 0.99, in which the friction parameter 0 is for complete adhesion and 0.99 is for nearly complete slide. The variation of the maximum shear stress under different conditions of friction parameters is shown in Figs. 7 and 8.

As shown in Figures 7 and 8, the change of the bonding condition between stress absorption layer and the upper and lower layers affects the maximum shear stress between itself and the bottom layers of asphalt directly. With the increase in friction parameters between asphalt layer and stress-absorbing layer, the maximum shear stress

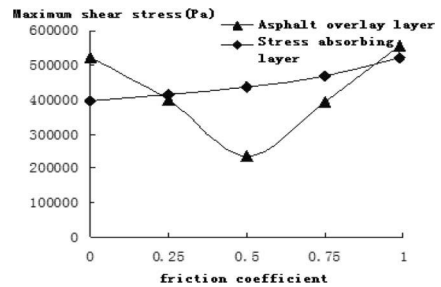


Figure 7. Maximum shear stress at the bottom of the layer under the condition of friction parameter of different asphalt overlay stress-absorbing layer.

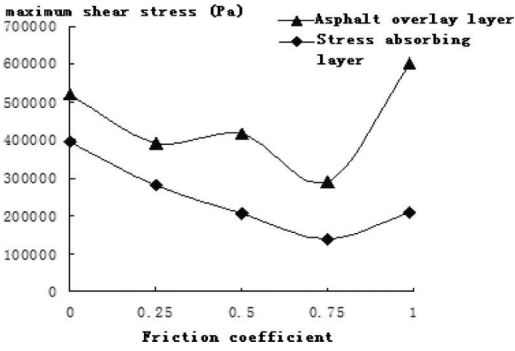


Figure 8. Maximum underlying shear stress between cement pavement and stress-absorbing layer under the condition of different friction parameter.

of rubber asphalt overlay will first decrease and then increase with the increase of friction parameters, while the maximum shear stress of the bottom stress absorption layer increases with it. The good bonding state of asphalt overlay layer can reduce the maximum shear stress of the stress absorption layer and prevent the premature failure of the stress absorbing layer. Although the change trend of asphalt overlay was first reduced and then increased, reducing the maximum shear stress of asphalt overlay by reducing the bonding degree between layers is not suitable, because at the beginning of building, the initial bonding state between the layers is not ideal. At the same time, over time, the bonding condition of interlayer will be gradually reduced, so it should not be the measures to prevent premature to enter the maximum shear stress growth. The bonding condition between the stress-absorbing layer and the cement pavement has a significant impact on the maximum shear. The maximum shear stress of rubber asphalt overlay and stress absorption layer bottom will first decrease and then increase with the increase of friction parameters. And for the asphalt layer bottom, completely sliding and in relatively complete bonding condition, the maximum shear stress increases by 15.23%, which also proves that we cannot reduce the maximum shear stress by reducing the adhesion of the layers.

2.3 Effect of different stress-absorbing layer thicknesses on the maximum shear stress of the bottom layer

BISAR is used for the mechanical analysis of rubber asphalt overlay (stress-absorbing layer) under different conditions of stress-absorbing layer thickness.

The thicknesses were 0, 0.6, 0.7, 0.8, 0.9, 1.0 and 1.5 cm. It can be found from calculation that when

we do not set stress-absorbing layer, the maximum shear stress of the asphalt overlay bottom layers is 5.959×10^5 , and the calculation results of bottom shear stress when settings with other different thickness of stress absorption layer are as shown in Figure 9.

As it can be seen from Figure 9, with the increase in the thickness of stress-absorption layer, the maximum shear stress between asphalt overlay and the bottom stress-absorbing layers decreases continuously, the maximum shear stress decreases by 12.59% when stress absorbing layer thickness is set at 0.6 cm. When the thickness is 1.0 cm, it decreases by 23.55%, and when the thickness is 1.5 cm, under the maximum shear stress, it decreases by 32.01%, which proves that the thickness effect is the effective measure to reduce the maximum shear stress of asphalt pavement; however, this effect increases with the thickness gradually. At the same time, for rubber powder-modified asphalt stress-absorbing layer, its thickness is 0.6–1.0 cm, and in the actual use of the process, because of the influence of Webster effect, the lower layer of asphalt will form a certain thickness of the oil-rich layer, the actual reduction effect of the maximum shear stress is greater than the one of calculation results. Therefore, the rubber powder-modified asphalt stress-absorbing layer is an economical and reasonable anticracking measure. In addition, from the variation of the maximum shear stress of absorption bottom layer in different thickness, it can be seen that increasing thickness cannot effectively reduce the maximum shear stress of stress-absorption layer. When the thickness of the stress-absorbing layer increases from 0.6 to 1.0 cm, the maximum shear stress decreases by only 3.44%. When the thickness of the stress-absorbing layer increases to 1.5 cm, the maximum shear stress decreases by only 7.5%. Therefore, the increasing of thickness of the stress-absorbing layer cannot effectively reduce the maximum shear stress by itself, which requires the stress-absorbing layer material's good elastic recovery, and the rubber powder asphalt has

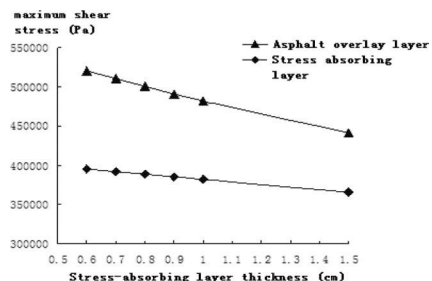


Figure 9. Bottom layer maximum shear stress under different thicknesses of stress-absorbing layer.

a high modulus and a high viscoelasticity trait, which is in line with this request.

3 CONCLUSION

1. By using BISAR mechanical calculation software, we calculated and analyzed the rubber asphalt compound pavement shear stress of different interlayer adhesive states, and under the condition of stress-absorbing layer thickness, we knew that the adhesive states of stress-absorbing layer, rubber asphalt surface layer, and underlying cement pavement have a direct impact on the distribution of shear stress of asphalt overlay and stress-absorbing layer, and the maximum shear stress of asphalt layer bottom will first decrease and then increase with the increase in friction parameters. Therefore, the bottom layer should have good adhesion to prevent the premature entry of shear stress growth period.
2. With the increase in the thickness of the stress-absorbing layer, the maximum shear stress of asphalt overlay is significantly reduced, but the effect of the maximum shear stress of the stress-absorbing layer is not obvious.

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