ON ORTHOTROPIC STEEL DECK PAVEMENT OF SUEZ CANAL BRIDGE

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The Suez Canal Bridge was constructed through a grant aid from Japan and completed in September 2001. Stone Mastic Asphalt pavement (SMA) was adopted over the orthotropic steel deck of this bridge because SMA was utilized for the orthotropic steel deck in Japan and SMA did not need any special machines, which were indispensable for the Gussasphalt Pavement (nonporous) mastic asphalt pavement) and were unavailable in Egypt. After the bridge opening, however, hair cracks on pavement began to appear from June 2002 due to overloading of vehicles whose axle weights sometimes exceeded 25t. Upon advice from Japan, the General Authority for Roads, Bridges and Land Transport (GARBLT) limited the axle weight of vehicles to 13t. In September 2003, after inspection of the bridge, the bridge was handed over to Egypt. In 2011, a study team from Japan International Cooperation Agency (JICA) was dispatched to investigate the condition of the pavement and the steel deck. Although the pavements were heavily cracked, no fatigue cracks on the orthotropic steel deck were found. The steel deck surfaces were investigated and rusts were confirmed. The thickness reduction was measured. The average largest thickness reduction was 0.5mm. As seepage water was confirmed on the steel deck, 20 water monitors were embedded. The team recommended the repavement using the Gussasphalt Method as it was practically the only one method applicable at present in Japan. In 2016, GARBLT decided to repave the bridge using its own method and actually repaved the bridge. JICA experts could not give any advice because there were no related experiences in Japan. However, this pavement was deemed viable. Before the repavement, the water monitors were checked and seepage water was confirmed. After the removal of the pavement, all of the steel deck surfaces were investigated.

**Key Words**: SMA pavement (stone mastic asphalt pavement), cracks, orthotropic steel deck, Suez Canal Bridge, resin-bonded surfacing

1. SUEZ CANAL BRIDGE

The Suez Canal Bridge is a cable-stayed bridge with span arrangement of 163+404+163=730m. On each side of the span there are two intermediate piers. This bridge secures a very high navigational clearance of 70m so that on both sides of this cable-stayed bridge, very high concrete piers continue for a considerable length. This bridge was constructed through a grant aid from Japan. The bridge was opened in October 2001 as a toll road. The Suez Canal Bridge is shown in Photo 1 and the position of this bridge is shown in Fig.1. The bridge is between Port Said and Ismailia in Qantara town.
The bridge is situated in or near the subtropical desert biome. The climate chart is shown in Fig.2. The weather is dry and the total annual precipitation averages 37mm. In winter from November to March, there is a little rain. However, it is not uncommon to observe fogs in the early mornings during winter.

2. PAVEMENT OF SUEZ CANAL BRIDGE

SMA was adopted for the base course of the pavement of the Suez Canal Bridge. SMA and the Gussasphalt method were compared for the orthotropic steel deck pavement and SMA was selected. SMA can be constructed without any special machines but the Gussasphalt need special machines to mix, to transport, and to finish. These special machines were not available in Egypt. The thickness of the pavement was decided as 8cm (4cm×2 layers), which was thicker than the usual bridge deck pavements, considering the 2cm high remains of hanging hooks on the steel deck. The steel deck was paved between June and August 2001. The structure of the pavement is shown in Fig.3. The adhesive layer consisted of three layers. Two layers of chloroprene rubber solvent adhesive, 0.3L/m² for the first layer and 0.4L/m² for the second layer were applied. As the third layer, asphalt rubber solvent adhesive of 0.2L/m² was applied.

The surface of the steel deck plate was sandblasted as other blasting methods were not available. The adhesives were applied immediately after the sandblasting. SMA of the binder layer and the dense-graded asphalt of surface layer were constructed smoothly and the test results of these two layers were satisfactory. The coarse aggregate available at the site and used for SMA belonged to one kind of limestone. The aggregate had a higher water absorption rate but satisfied other material criteria. The plant mix-type asphalt modifiers were imported from Japan.

(1) Surface course

It was planned to adopt the dense-graded asphalt mixture (13mm) for the surface course, which had the same grading as the surface course of the approach concrete viaduct and which was planned to be constructed by the Egyptian contractor. Later, however, it was decided to adopt the Type 2 polymer-modified asphalt mixture (13mm).
asphalt whose specification is shown in Table 1, to achieve higher flow-resistance. The plant mix-type asphalt modifiers were used.

(2) Base course

SMA is inferior to the Gussasphalt method in the following points: the deflection followability and the adhesion to the steel deck surface. To improve these points, the improvement of SMA was tried in Japan. After the examination using Japanese materials, Egyptian asphalt and aggregates were imported and tested. As a result of the improvement of the asphalt mixture, the dynamic stability of 1,580 cycles/mm and the bending breaking strain of 6.1x10^{-3} cm/cm were obtained, and which were satisfactory. The finer-grade aggregates were increased to achieve better deflection followability. Also the Type 2 polymer-modified asphalt by plant mix modifier was adopted to increase the flow resistance. To increase the durability and to increase the film thickness of the binder, plant fibers were added. The grading curve of Suez SMA is shown in Fig.4. As a reference, the median particle size of SMA of Japan Highway Corporation(JH), which was used for the waterproofing pavement, is also shown. In addition, the particle size of the present SMA of Nippon Expressway Company (formerly JH, NEXCO, 2015) is also shown in Fig.4 with the median particle size. The pavement of the Suez Canal Bridge was constructed in August 2001. The physical property values of Suez SMA are shown in Table 2. Eight cores were sampled at the site and the degree of compaction was measured. The results as shown in Table 3, are satisfactory. In the paper, there were no data of measured adhesion between the pavement and the steel deck. Data were unavailable from the contractor either, as the construction was done about 15 years ago.

3. HISTORY OF PAVEMENT AFTERWARDS

The history of the pavement of the Suez Canal Bridge is shown in Table 4. This table shows how the bridge was influenced by the pavement method change in Japan. The reason for the change is as follows: SMA of this bridge was constructed when SMA was experimentally adopted and the standard of SMA was specified in Japan. After the construction of SMA, however, comparatively earlier forms of damage were reported in Japan. Therefore at around 2007, SMA was no longer adopted for the orthotropic steel deck pavement in Japan. If this bridge had been constructed later than 2007, SMA might have not been adopted. Also if this bridge were located in Japan, SMA pavement would have been repaved using the Gussasphalt method much earlier. The Suez Canal Bridge was opened in October 2001 as a toll road. In May 2002, the ferry service, which had been operated before the bridge opening and utilized by heavier trucks, was stopped and the bridge began to be fully utilized. Soon after the ferry service closure, hair cracks on the pavement began to appear in June 2002. This was considered to have been caused by the passage of heavier trucks whose axle weight sometimes exceeded 25 t. The consultant and

| Table 1 Standard characteristics of Type 2 polymer-modified asphalt |
|---------------------------|-----------------|-----------------|-----------------|-----------------|
| Softening point (°C)      | More than 56.0  |
| Ductility (15°C)          | More than 30    |
| Toughness(25°C)           | More than 8.0   |
| Tenacity(25°C)            | More than 4.0   |
| Penetration Test(25°C) 1/10mm | More than 40  |
| Mass change rate %        | Less than 0.6   |
| Penetration Residual Rate after a thin membrane heating test % | More than 65 |
| Flash Point (°C)          | More than 260   |

| Table 2 Physical property values of Suez SMA |
|---------------------------------------------|-----------------|-----------------|-----------------|-----------------|
| Items                                       | Standard Value  | Largest Value   | Smallest Value  | Average Value   |
| Result of Asphalt Extraction Test           |                 |                 |                 |                 |
| 2.36mm (%)                                  | 35.6            | 35.9            | 33.9            | 35.2            |
| 0.075mm (%)                                 | 10.0            | 10.2            | 9.1             | 9.7             |
| Amount of Asphalt (%)                       | 6.5             | 6.58            | 6.45            | 6.53            |
| Marshal Stability Test                       |                 |                 |                 |                 |
| Density (g/cm³)                             | —               | 2.336           | 2.334           | 2.335           |
| Air Void (%)                                | ≤2              | 2.8             | 2.7             | 2.8             |
| Voids Filled with Asphalt (%)               | 75.9            | 84.3            | 83.7            | 84.0            |
| stability (kg)                              | ≥2500           | 1,166           | 1,050           | 1,111           |
| stability (kN)                              | ≥24.90          | 11.4            | 10.3            | 10.9            |
| Flow value (1/100cm)                        | 20-60           | 53              | 47              | 50              |

| Table 3 Degree of compaction                  |                  |                  |                  |
|---------------------------------------------|-----------------|-----------------|-----------------|
| Construction Date                           | Degree of Compaction (%) | Construction Date | Degree of Compaction (%) |
| Aug.2nd, 2001                               | 96.0            | Aug.6th, 2001   | 97.2            |
| Aug.4th, 2001                               | 97.0            | Aug.9th, 2001   | 97.2            |
| Aug.5th, 2001                               | 98.4            | Aug.11th, 2001  | 97.8            |
| Aug.6th, 2001                               | 98.9            | Aug.12th, 2001  | 96.0            |
the contractor asked GARBLT to restrict the axle weight of heavy trucks properly. GARBLT restricted the axle weight up to 17t in July 2002 and further lowered the limit up to 13t in September 2002. (This was still slightly higher than the 10t upper limit of Japan.) In response to this situation, the defects liability period was extended for one more year, until September 20th, 2003. The pavement condition was observed and cracks were repaired by the contractor. The defects liability period ended with the donation of pavement repair materials for the next three years.

From October 2010 to November 2011, “Follow-up

Table 4 History of pavement of Suez canal bridge and its surrounding circumstances

| Year | Event |
|------|-------|
| 2016 | Advice on pavement removal and repavement. |
| 2015 | Bridge reopened only at night from 18:00 to 1:00. |
| 2015 | GARBLT decided to have the steel deck repaved by Egyptian contractors and asked JICA for assistance. |
| 2013 | Advice on pavement removal and repavement. |
| 2012 | As a result of this Follow-up Cooperation Study, repavement of the steel deck by Gussasphalt Method was recommended to be most suitable. |
| 2011 | The Follow-up Cooperation Study (Pavement on Steel Deck) |
| 2011 | During FC Study, Pavement condition was surveyed. |
| 2011 | The Follow-up Cooperation Study for the Substructures (Substructure FC Study hereinafter) |
| 2010 | Advice on pavement removal and repavement. |
| 2007 | At around this time, SMA was no longer adopted for the orthotropic steel deck pavement in Japan because some problems were found on SMA adopted for the steel decks, such as pot holes, cracks, etc. |
| 2003 | Three years worth of pavement repair materials were donated. |
| 2003 | 20th Sept. End of defects liability period. |
| 2002 | Ferry Service resumed. |
| 2001 | Opening of Bridge |
| 2001 | SMA Pavement on Suez Canal Bridge |
| 2001 | Design and Construction Guide of SMA Pavement on Orthotropic Steel Deck, was implemented by Hanshin Expressway |
| 1992 | Hashin Expressway Company experimented SMA pavement on orthotropic steel deck.4) |

During FC Study, Pavement condition was surveyed.

2007

The Follow-up Cooperation Study (Counterpart Training)
cooperation study on the project for construction of
the Suez Canal Bridge (Substructure)” was imple-
mented. This was because some damage on the con-
crete piers constructed by the Japanese contractor
were found and to repair the damage, this study was
implemented. During this study, the pavement con-
tion was checked in May and August 2011 by the
JICA Study Team. The condition of the pavement
was worse than in September 2003, when the defects
of pavements were checked, although GARBLT con-
tinuously imposed the axle weight limitation and re-
paired the pavement cracks once a year. Based on this
observation, GARBLT requested JICA for a follow-
up cooperation study (pavement on steel deck) to
check the condition of the pavement and the steel
decks. Advice for the necessary countermeasures was
requested. This cooperation study began in October
2011 and lasted until March 2013. During this period
and from November to December 2011, the pave-
ment and the orthotropic steel deck were examined
precisely. Although the pavement was heavily
cracked, no fatigue cracks of the steel deck were
found. As a result of this study, the early repavement
of the bridge deck using the Gussasphalt method was
recommended as SMA was almost no longer adopted
for the orthotropic steel deck pavement in Japan.

At the same time, it was recommended to send
JICA experts for the construction of the Gussasphalt
pavement and the reinforcement of the orthotropic
steel decks when GARBLT would repave the bridge,
as it was expected that the repavement of the bridge
deck would take some time after the recommenda-
tion.

In July 2013, the Suez Canal Bridge was closed to
traffic due to the political turmoil in Egypt. After the
above recommendation, GARBLT decided to pave
the bridge deck using its own fund and its own
method, rather than the Gussasphalt method. In 2016,
“Follow-up cooperation study on the project for con-
struction of the Suez Canal Bridge (Counterpart
Training)” was implemented based on the request
from GARBLT to support the repavement works of
the Suez Canal Bridge by GARBLT.

In January and September 2016, JICA experts
were dispatched and the steel deck condition was
checked. As for the pavement using the GARBLT
method, JICA experts could not comment because
there were no examples of that kind in Japan.

4. RESULT OF SITE SURVEY OF PAVE-
MENT IN 2011

The whole 730m bridge length was inspected pre-
cisely and recorded. It was observed that 95% of the
pavement area contained cracks. Approximately 70% of
the bridge surface area contained cracks whose
widths were more than 10mm. The following facts
became clear:

1) The outside lane of the south side lane, i.e., the
east bound lane, was the most severely cracked. The
heavier trucks moved from the west, Cairo side,

(Fig.5 Locations of detailed inspection.
Fig.6 Relation between truck tire positions and U-Ribs.)
to the east, Sinai Peninsula side. The eastbound lanes or the south side lanes had the largest total weight of the passing vehicles. This might be the reason for larger damage.

2) The inside lane of the north side lane, i.e., the westbound lane, was less damaged.

3) The driving lanes, i.e., the outside lanes, were more damaged.

4) The driving lanes had more traffic than the passing lanes.

5) From the observation of the cracks, the occurrence of cracks was proportionate to the total weight of the passing vehicles.

Six severely damaged areas were selected and inspected precisely. The areas are shown in Fig.5. The size of each area was 6m long and 4m wide. The inspected items were as follows: The floating condition of the pavement by hammer test; the width, depth, and length of cracks; and the flatness of the pavement. The relation between the tire position of trucks and the U-Ribs of the orthotropic steel deck is shown in Fig.6. One of the inspected results (N1 position in Fig.5) is shown in Fig.7. From this figure, extensive cracks can be confirmed. The hair cracks found in June 2002 are shown in Photo 2. From the comparison of Fig.7 and Photo 2, the progress of the deterioration can be confirmed although the positions of Fig.7 and Photo 2 may not be the same. The Suez Canal Bridge was opened in October 2001. Quite soon after the opening, hair cracks were found in June 2002. From the observation of the SMA pavement on the steel decks of Hanshin Expressway Company in Japan, pot holes and delamination occurred mostly in the comparatively earlier stage, i.e., within one to two years after the construction. Pavement cracks...
It was reported in another paper that about 50 cracks occurred along the bridge axis direction. In compression. The same kind of cracks occurred along the webs of U-ribs, not over the webs of U-ribs. This might mean that the cracks occurred under the compression only. This fact is shown again in Sec.7.

The largest observed crack had a width of 40mm and a depth of 80mm; this meant that the crack reached the steel deck plate. As shown in Fig.7, cracks seemed to occur over the steel deck between the webs of U-ribs, not over the webs of U-ribs. This might mean that the cracks occurred under the compression. The same kind of cracks occurred along the bridge axis direction. In Photo 3, a crack occurred between two diaphragms. This might have occurred under the compression. The following reason may explain why cracks occurred seemingly under the compression.

It was reported that the longitudinal cracks occurred on the orthotropic steel deck pavements, which was one of the typical characteristics of this SMA pavement. It applies to this pavement, too. It was reported in another paper that about 50 bridges with orthotropic steel deck pavements of the Metropolitan Expressway Company in Japan were investigated and out of 50 bridges, a few contained longitudinal cracks not only over the U-rib webs but also between the webs of U-ribs. In this case, cracks occurred both above the webs and between the webs. It was also reported that the cracks occurred even only under compressive strains, which was confirmed by the repetition bending test that gave only compressive strain to the test specimens.

In the case of the Suez Canal Bridge, however, cracks occurred seemingly only between the U-rib webs. Cracks did not occur over the webs. This was quite different from the observations of other bridges in Japan. The fact that cracks can occur even under compression only is proven by the above paper. The cracks of the Suez Canal Bridge occurred over places where the deformation was larger, i.e., between the U-rib webs or between the diaphragms as shown in Photo 3. This might suggest that these cracks were caused by the larger deformations. With the larger deformations, a larger friction between the pavement bottom and the steel deck surface occurred. This friction might have cut out the adhesion between the pavement bottom and the steel deck surface more than over the webs. Without the adhesion to the steel deck, SMA pavement might have cracked more easily. This could explain why cracks occurred between the U-rib webs but not on the webs. This fact is shown again in Sec.7.

In places where larger cracks were found, cores with 100mm diameter were bored to examine the cracks and the adhesion to the steel deck (Photo 4). Six places were selected. The adhesion strengths were measured at the spot. The average measured adhesion strength was 0.113N/mm². This value was much smaller than 0.6N/mm², which was specified as the lowest allowable adhesion strength between a concrete bridge deck and a pavement (9). The average measured adhesion strength 0.113N/mm² was almost equal to the weight of cores and this meant that the adhesion was almost lost. This might be due to the deterioration of the adhesive layer by the seepage water from the cracks. All of the cracks inside the sampled cores reached the steel deck.

Seven square areas of pavement were removed to inspect the condition of cracks and the adhesive layers (Fig.8). They consisted of three places (L) of 1 m × 1m, two places (S) of 0.3m × 0.3m, and two places (SA) of 0.15m × 0.15m. Two places (SA) were selected from the healthy areas. One of the removed results is shown in Photo 5. As a result of this inspection, the following facts became clear. It was highly possible that the cracks more than 10mm wide reached the steel deck. Seepage water was confirmed on the adhesive layer. The adhesion strength between the base course and the steel deck on the cracked area did not satisfy the necessary strength. It was highly possible that 70% of the pavement area deteriorated considerably. The adhesive layer also deteriorated so that the whole pavement needed to be replaced. (In the case of the Gussasphalt method, if the Gussasphalt layer, i.e., the base course, is healthy and this is generally the case, only the surface course is renewed.)

In Photo 6, the pavement of the healthy area (SA2) was removed to examine the condition. The result showed that the pavement was healthy from the steel deck to the surface. The healthy areas were near the center median and almost no tires had run over these areas. This was the reason that the areas had remained healthy.
5. RESULT OF SITE SURVEY OF STEEL DECK IN 2011

The surfaces of the steel deck after the removal of the pavement were investigated. Then the deck plate was visually inspected from the inside of the box girder to determine the places for the Magnetic Particle Test (MT) and the Ultrasonic Test (UT).

(1) Steel deck surface inspection

The areas shown in Fig. 8, i.e., L1, L2, L3, S1 and S2, were investigated. In the L1 area, after the removal of the pavement, red brown substances, probably rust, remained (Photo 7). Only the western half of L1 was rusty and the plate thickness decreased. L2 before the removal of pavement is shown in Photo 8. The extensive cracking can be observed. In L2, a rust layer of about 0.5mm to 1mm stuck to the steel deck.

Table 5 Average deck plate thickness (mm).

| L1 (West Half only) | S1 | L2 | S2 | L3 |
|---------------------|----|----|----|----|
| 11.7                | 11.6| 11.5| 11.7| 11.5|

### References

- Photo 5: Cross-section of pavement after pavement removal (L2).
- Photo 6: Removal of healthy pavement area (SA2).
- Photo 7: Removed pavement (L1) and red brown substances.
- Photo 8: L2 and S2 before pavement removal.
- Photo 9: L2 after removal of rust.
- Photo 10: Enlarged surface of L2 after removal of rust.
The deterioration of L2 and L3, which were on the south side and the eastbound lane, proceeded more than L1, which were on the north side and the westbound lane. L2 after the removal of rust is shown in Photo 9. Its enlargement is shown in Photo 10.

From the surface observation of five places, the following facts became clear: Seepage water under the pavement and the rust were confirmed in all places. The degree of rust differed from one place to another. To stop the steel deck deterioration, the deck needed to be shot-blasted and totally repaved, including the waterproof layer.

The thickness of the steel deck was measured by ultrasonic thickness gauge to detect the thickness reduction. The original thickness was 12mm. Five to ten points inside one place were measured and the average thickness was calculated. The results are shown in Table 5. (For L1, only the west half was rusty and measured.) The largest thickness reduction of 0.5mm was found on L2. Smaller dents due to corrosion were found but ignored from the thickness measurement. From the structural point of view, the thickness reduction may not be a large problem at present, but can influence the fatigue strength and the deformation performance in the future. The design stress variations of the U-rib were calculated based on the deck plate thickness change. The results are shown in Table 6. The stress change is below 1% even if the plate thickness becomes 11.0mm and there is still a large margin to the allowable stress of $\sigma_a = 137.2\text{MPa}$ (=1400kg/cm²).

### Magnetic Particle Test (MT) and Ultrasonic Test (UT)

The inside of the box girder were visually inspected to check the condition and to determine the places for the MT and UT. The welding connections of the deck plate and the diaphragms or the deck plate and the lateral ribs, which were directly below the pavement removal areas (L1, L2, L3, S1, S2), were visually inspected from the inside of the girder. Also the welding beads of R7 and R8 U-ribs along the whole bridge length, which were directly below the tires of running trucks, were visually inspected. No fatigue cracks were found during the inspection.

MT was tried on the deck plate where the pavement was removed and the welding beads directly below the same areas. No fatigue cracks were found.

Sometimes fatigue cracks are initiated from the roots of welding beads inside the U-ribs as shown in Fig.8. It is very difficult to detect these cracks by visual inspection. UT is the only method that can detect this fatigue crack before it penetrates the steel deck and appears above it. UT test was applied on the welding beads of U-ribs and the steel deck under L1, L2, L3, S1 and S2 areas, but no fatigue cracks were found.

The same U-ribs were tested by hammer to examine the sound. If fatigue cracks reach the surface of the deck plate, then water, sand, and rust may accumulate inside the U-ribs. This can be detected by hammer test. No abnormal sound was detected.

No indication of fatigue cracks was found during the investigation. This might be because the 13t axle weight limit was strictly enforced although in the first year of the bridge opening, heavier trucks passed the bridge. Another fact was that the bridge had been in service for only about 10 years.

After the examination of pavement cracks and the inspection of the steel deck, the pavement cracks along the entire bridge were repaired because it was expected that it would take some time before the repavement.

### 6. INSTALLATION OF WATER MONITOR IN 2012

During the examination of pavement cracks in 2011, seepage water above the steel deck was
confirmed. To monitor this water, 20 water monitors shown in Fig.10, were installed in May 2012 along the whole bridge length, near the edge of pavements so as not to interfere with the traffic. The monitors were observed by GARBLT and JICA experts afterwards. Water was confirmed even in the summer when there were no rains nor foggy days. The water monitor observation will be discussed more in the later section.

7. INSPECTION OF STEEL DECK SURFACE AND REPAVEMENT IN 2016

In June 2015, GARBLT notified JICA that they would remove the pavement and repave the steel deck and asked JICA’s assistance for the inspection of the steel deck. The JICA Study Team composed of two engineers was dispatched to the site in January and August 2016.

(1) Inspection of steel deck surface

The bridge deck was investigated in November to December, 2011. After one year and a half, in July 2013, the bridge was closed due to political turmoil. Therefore it was expected that new fatigue cracks would not be found. To repave the deck, a whole old pavement needed to be removed. The JICA Study Team could have inspected the whole bridge deck surface, which was a rare occasion. First, the pavement of the north side and outside lane near the west pylon area was removed and the steel deck surface was inspected. As shown in Photo 11, on both edges of outside lane, an extensive corrosion was confirmed. At the middle of the outside lane, the condition was better. The left side corrosion might have corresponded to the longitudinal cracks of the pavement, water might have permeated into the steel deck surface and corrosion might have been formed. The right side corrosion might have caused by the water coming from the pavement end. This will be explained more in Photo 13. The worst corroded area was confirmed near the west pylon as shown in Photo 12. The pavement above this area might have cracked heavily and the water might have permeated largely. No lines of rust, which could mean fatigue cracks that initiated from the welding beads of U-ribs and reached the deck surface, were confirmed. Then the pavement removal work proceeded further.
The fatigue crack-prone places, welding beads of U-ribs, etc. on the back side of the steel deck were also visually inspected from the inside of the box girder, but no signs of cracks were found. **Photo 13** shows a corrosion near the curb. This might be due to the water that permeated through the gap of the pavement and the curb, although it was sealed as shown in **Fig.3**. The original molding joint sealant might have not worked well. **Photo 14** shows that the same situation was observed near the center median. If the deck plate was corroded, the back side of pavement became red brown because of the rust. In **Photo 15**, a healthy deck plate near the center median is shown. The back side of the corresponding pavement was black. Although this place was near the center median, the permeated water did not reach this place probably because of the healthy adhesion between the pavement and the steel deck, as tires rarely passed over this area, which was the same situation as in the SA2 area shown in **Photo 6**. In **Photo 16**, one piece of reversed pavement is shown. Although this pavement seemed to be healthy, some water permeated, reached the deck, and corroded some parts of the steel deck surface. In **Photo 17**, the pavement removal work is shown. The workers turned over the pavement by levers. In the case of the Gussasphalt, the adhesion between the pavement and the steel deck was reported to be more than 1.4MPa\(^9\). If it was assumed that the adhesion of this SMA pavement of the Suez Canal Bridge was also 1.4MPa with area of one square meter, then the overturning force of the pavement became almost 1.4MN (≈143tf). Thus it was completely impossible to turn over the pavement by manpower. This pavement overturning work proved the weak adhesion of this pavement. In **Photo 18**, the back sides of the removed pavement pieces are shown. From this photo, the area of corrosion of the steel deck can be roughly estimated. About a half of the bridge deck might have been corroded. After the removal of the pavement, the thickness of the whole steel bridge deck plate was measured. The North Lane was measured in February 2016 and the South
Lane in August 2016. Four points of the North Lane and four points of the South Lane as shown in Fig.11 were measured. The total bridge length was 730m. From the west end to the east end, the bridge deck was measured at about 25m intervals. The worst corrosion of the North Lane was found at C point and 5m from the east end of the bridge. The thickness was 11.16mm. The original thickness was 12.0mm. The second-worst corrosion was found at D and 58m from the west end and on the side span (Photo 19). The thickness was 11.21mm. The measurement of this corroded section is shown in Photos 20 and 21. As the surface was covered with rust, the rust needed to be removed first. Then the surface was polished by power brush and a part of the surface was ground flat so that the correct thickness could be measured. The uneven surface could not be measured correctly. After this measurement, uneven surfaces, which showed unreasonable thickness values, were ground so as to have small flat areas for the measurement. The result of thickness measurement is summarized in Table 7.

As shown in the table, the corrosion of the south side was worse, which could be expected because of the worse cracks of pavement. The south side minimum thickness of 11.00mm was found at B point and 150.5m from the west end. As shown in this table,

| Table 7 Average and minimum deck plate thickness (mm). |
|------------------------------------------------------|
| **Average Thickness**                                |
| North Side                                           | 11.84mm |
| South Side                                          | 11.68mm |
| Total                                               | 11.75mm |
| **Minimum Thickness**                               |
| North Side                                          | 11.16mm |
| South Side                                         | 11.00mm |
the average thickness is 11.75mm. This value may be still admissible from the calculation of Sec.5 (1). However, careful observation is indispensable in the future. After the thickness measurement, the steel deck was sand-blasted to remove rusts and painted with Zinc-Chromate, rust proof paint, so that further corrosion could be stopped. This will be discussed later. During the pavement removal work, the remaining hanging hook on the steel deck and the steel deck itself were slightly cut by pavement cutting machine as shown in Photo 22. The study team requested the contractor to smoothen the cuts and not to cut the steel deck again. The cuts were smoothened to avoid stress concentration. The smoothened result is shown in Photo 23.

(2) Pavement cracks
The repaired cracks of pavement were observed before and during the pavement removal work. The cracks were repaired by the JICA Study Team in December 2011 after the investigation of pavement and the orthotropic steel deck. The largest repaired crack was on the South Lane and close to the center of the center span, which is shown in Photo 24. Other repaired cracks are shown in Photo 25. As the cracks were repaired, it became clearer that the cracks were aligned in the same direction. The distances between the curb and the lines were measured. The distances were 1440mm, 1760mm, and 2080mm respectively. These distances corresponded to the centers of the U-rib webs (Fig.12). The line of cracks happened over the empty space between the webs of U-rib. These cracks might have occurred under the compression as explained earlier in Fig.7.

As seen in Sec.5, under the cracks, seepage water and the rust were confirmed. Under the cracks shown in Photo 25, the steel deck must have been rusty, too. The pavement cracks might be the cause of the steel deck deterioration. Rainwater and dew water penetrated into the steel deck surface and deteriorated the steel deck. If the pavement was cracked in the manner shown in Fig.7, the steel deck must have been as rusty as the steel deck plate shown in Photo 12. Although Fig.7 and Photo 12 were not exactly the same place, they were very close to each other. Another factor for the steel deterioration was the seepage water from the pavement edges into the steel deck surface. This water deteriorated the edges of steel deck surfaces as shown in Photos 13 and 14. In the area where no seepage water came from the pavement edges, the steel deck remained healthy as shown in Photo 15. Over the road shoulders, tires rarely ran and pavement cracks did not develop. The center areas of the road lanes also received less tire pressure and remained comparatively healthier. This might be the reason why the center areas remained healthier.

(3) Water monitoring
As mentioned in Sec.6, 20 water monitors were
installed. Their positions are shown in Fig. 13. The observed results at noon time on January 31st, 2016 are shown in Table 8. Slight rainfalls were observed before this day but not on this day itself. It was foggy in the morning. The monitors whose caps were lost, were filled with wet sand (Photo 26). P13, P15, P17, and P19 were already removed. For P3, P6, P9, and P11, caps remained and wet bottoms were observed. The rubber plate bottoms were swollen and could be easily broken by hand and a rusty deck plate appeared (Photo 27). For P12, P16, P18, and P20, higher water level was observed (Photo 28). In this area, all of the bottom of the pavement must have been under the water. The removed P13 monitor was examined. In Photo 29, a swollen rubber bottom plate was observed. As the surrounding pavement bottom was red because of the rust and the existence of water could be detected. In Photo 30, the rubber plate was removed and the monitor structure could be understood easily. In Photo 31, a rusty deck plate below P13 is shown. The rubber plate was placed to eliminate water but it was not effective. After the deck plate preparation, it might have been better to paint the deck plate instead of the rubber plate. Although January was in winter and in the rainy season, the average monthly precipitation was only 7mm (Fig. 2) and the

| East     |                      |
|----------|----------------------|
| P1       | No cap. Filled with wet sand |
| P2       | No cap. Filled with wet sand |
| P3       | Cap remains. Some water. |
| P4       | No cap. Filled with wet sand |
| P5       | No cap. Filled with wet sand |
| P6       | Cap remains. Wet rubber bottom. |
| P7       | No cap. Filled with wet sand |
| P8       | No cap. Filled with wet sand |
| P9       | Cap remains. Wet rubber bottom. |
| P10      | No cap. Filled with wet sand |
| P11      | Cap remains. Some water. |
| P12      | Cap remains. High water level. |
| P13      | Removed with pavement. Rusty Deck. |
| P14      | No cap. Filled with wet sand |
| P15      | Removed. Rusty Deck. |
| P16      | Cap remains. High water level. |
| P17      | Removed. |
| P18      | Cap remains. High water level. |
| P19      | Removed. |
| P20      | Cap remains. High water level. |

Table 8 Observed results of water monitors.
road surface was almost always dry, it seemed that only the bottom of the pavement of this bridge was wet. This may be due to two reasons. One reason is that the rain water penetrates the pavement through its cracks and the water stays there without evaporation. The other reason is that the dew condensation on the pavement and its cracks. As shown in Photo 32, in winter, it often becomes foggy in the morning near the bridge. This means that the temperature is below the dew point. Therefore dew drops are also formed on the surface and the inside of cracks of the pavement. These drops may accumulate and stay at the bottom of the pavement. In winter, there are rainfalls and foggy days, thus it is still understandable to some extent to find water accumulation under the pavement. In summer when there are no rainfalls nor foggy days, still some water under the pavement was confirmed by the JICA study team. This might be because of the large temperature difference between night time and day time, and dew formed on the pavement. Higher water level was observed at P12, P16, P18, and P20 compared to P9, P6, and P3 possibly for the following reasons: the morning mist was one of the major sources of water supply. The bridge was parallel to the east-west direction. The bridge was the highest at the center and descended down to both sides with 3.3% gradient. The mist occurred only in the morning. As the sun rose, the morning mist disappeared and the dew on the pavement surfaces evaporated due to the sunlight. The incidence angle of the sunlight, however, was different on the east side and the west side of the bridge because of the 3.3% road gradient. The dew on the east side evaporated more than on the west side because of the gradient. This might be the reason why a higher water level was observed at P12, P16, P18, and P20.

(4) Condition of girder

The inside of the box girder was dry owing to the dehumidifier installed after the completion of the bridge. Before the installation, the inside had been
humid and some parts had become rusty. This humidity might have come from the sea water of the canal. The box girder was inspected from the inspection vehicle. The whole surface of the box girder was covered with very fine sand (Photo 33), which probably came from the desert in the east side of the bridge. The west side of the bridge was irrigated by the water of the Nile River and the land surface was green. Only spot rusts, like the one shown in Photo 34, were found. The condition of the girder was quite good after about 15 years because of the dry weather as seen in Fig.2. The relative humidity was always below 60%. As a whole, the condition of girder surfaces were much better than those of the steel decks under the pavement. Only one severe corrosion was found on the wind fairing (Photos 35 and 36). The site welding beads was corroded. As the site welding might not be as good as the shop welding, the rugged surface of the beads might have collected more dew water and the welding beads might have become rusted. Although the wind fairing was not a structural member, it was recommended that GARBLT repair and repaint the coating as soon as possible.

(5) Repavement

The Gussasphalt method was proposed by the study team in 2012, as the SMA method was almost no longer adopted for the orthotropic steel deck pavement in Japan. However, the Gussasphalt method needs special machines, which are not available in Egypt. GARBLT proposed its own method, which is shown in Fig.14. According to Salah Mostafa of Polymar-Chemical Industries for Construction Co. in Egypt, which constructed the pavement of the Maasala Bridge, on the surface of steel deck, Zinc-Chromate Primer was applied to prevent corrosions. Over this primer, thick elastic mortar, a mixture of the tar epoxy and the polyurethane, was applied to fix small coarse aggregates consisting of dolomite, which is
one kind of limestone. Although dolomite was specified, any kind of aggregates could be substituted. The elastic mortar, which was actually used for the Masaala Bridge in Cairo City and saved separately at that time, is shown in Photo 37. An example to show the combination of aggregates and elastic mortar over a thin steel plate, is shown in Photo 38. This is called “resin-bonded surfacing.” This method seems to be often adopted by the French ODA projects. The same method was adopted for the Ba Baong No.1 Bridge in Cambodia, too, as a French ODA Project. In Cairo City, the Galaa Bridge near the International Airport is one example as shown in Photo 39. Another example is shown in Photo 40. The surfacing already deteriorated. After four to five years, it is believed that the resin-bonded surfacing would deteriorate\(^2\). The proposed method by GARBLT is to add a dense-graded asphalt mixture over this resin-bonded surfacing. The Maasara Bridge, an example of this type of pavement is shown in Photo 41. This bridge was constructed as a French project. This bridge might have adopted the orthotropic steel deck structure to achieve a shorter construction period. The steel deck consists of (about 3.5m)\(\times\) (8 to 10m) panels as shown in Photo 42. These panels were connected at the site. In Photo 41, lateral pavement repairs are visible at about 8 to 10m intervals and these repairs correspond to the connections of the two adjacent panels over the piers. In Photo 41, a longitudinal crack is also seen. This corresponds to the longitudinal connection of the two adjacent panels. The condition of the pavement was investigated and the result is shown in Photo 43. There were no rusts on the steel deck and the yellow color of Zinc-Chromate Primer still remained. The condition of asphalt pavement was also good. The Maasara Bridge was constructed in 1988 with resin-bonded surfacing. In 1994, the bridge was repaved using the pavement method shown in Fig.14. Since then, the bridge has never been repaved. In 2016, this pavement has kept its shape for more than 20 years. Although the amount of traffic on this bridge is not large, sometimes larger trucks pass over this bridge. As there were no experiences related to the Gussasphalt method nor special machines in Egypt, the Maasara Bridge must have been repaved first using the resin-bonded surfacing. The dense-graded asphalt mixture, which could be constructed by the Egyptian contractors, was then applied over the surfacing. This method turned out to be successful in the end. The reason for its success is explained in the next section. As this kind of pavement does not exist in Japan and the study team could not comment on this method, the team asked GARBLT to adopt this method on its own responsibility, although the possibility of achieving successful results seemed to be high. After the removal of the pavement and the investigation of the steel deck surface of the Suez Canal Bridge, the steel deck surface was sand-blasted to remove the rust, then Zinc-Chromate Primer was applied over the steel deck. The proposed pavement shown in Fig.14 was applied. The details of this pavement are shown in Table 9. The details of the mixture of tar epoxy and polyurethane were not available. In the pavement design, five alternatives with different bitumen contents, i.e., 4.5%, 5.0%, 5.5%, 6.0%, and 6.5%, were compared on April 20th 2016 and 5.5% (+0.25%) was recommended. In the trial mix, three alternatives with different bitumen contents of 5.50%, 5.75%, and 6.00% were compared on
May 28th, 2016. It was decided to adopt 6.00% to ensure higher durability and flexibility. The extraction test result (ASTM D2172) and the Marshal test result (ASTM D 1559) obtained from ACE Consultant, Egypt, which supervised the pavement construction work, are shown in Tables 10 and 11, respectively. In the actual construction, the bitumen content was between 5.80% (min) and 6.20% (max).

The size of coarse aggregate was mainly less than 9.5mm, which was comparatively smaller than those adopted in Japan. This size of coarse aggregate is often adopted for pedestrian pavements in Japan.

The sand-blasted steel deck plate is shown in Photo 44. The Zinc-Chromate Primer coated deck-plate is shown in Photo 45. The finished pavement of the North Lane is shown in Photo 46. This pavement needs to be monitored carefully for the next two or three years and if no cracks nor potholes are observed, then the effectiveness of this pavement shall

### Table 9 Pavement layers.

| Step | Layers |
|------|--------|
| 5    | 65mm thick dense graded asphalt mixture, \(D=12.5\)mm, 6% bitumen content (Result at site was 5.80% to 6.20%) and 1.7% Air Voids. |
| 4    | Tack coat, 0.3L/m² |
| 3    | 1 - 1.5cm thick polymer with 0.5-1mm sand filler was applied. Over this polymer, aggregate graded 2-7mm was scattered. This was the Elastic Mortar Layer. |
| 2    | Painting with polymer (mixture of tar epoxy and polyurethane, detailed spec. was unavailable.), 0.4L/m² |
| 1    | Zinc-Chromate Primer after sand blasting. |

### Table 10 Extraction test ASTM D2172.

| Sieve size (inch) | Cumulative Retained (gm) | Re-tained (%) | Passing (%) | Job Mix. Tolerance |
|-------------------|--------------------------|---------------|-------------|-------------------|
| a-WT. Of Sample before extraction (gm) | 1500.0 | b-WT. Of Sample after extraction (gm) | 1412.0 | c-WT. Of Bitumen (gm) | 88.0 | d-Percentage of Bitumen in Sample (%) | 6.23 |

| 1" | 25 | 0 | 0 | 100 | 100 |
| 3/4" | 19 | 0 | 0 | 100 | 95.0 | 100 |
| 3/8" | 9.5 | 63 | 4 | 95.5 | 92.2 | 100 |
| #4 | 4.75 | 529 | 37 | 62.5 | 58.3 | 66.3 |
| #8 | 2.36 | 804 | 57 | 43.1 | 43.3 | 49.3 |
| #30 | 0.600 | 1115 | 79 | 21.0 | 19.5 | 25.5 |
| #50 | 0.300 | 1229 | 87 | 13.0 | 14.0 | 20.0 |
| #100 | 0.150 | 1300 | 92 | 7.9 | 9.4 | 12.4 |
| #200 | 0.075 | 1368 | 97 | 3.1 | 4.0 | 7.0 |

### Table 11 Marshal test ASTM D1559.

| No. | 1 | 2 |
|-----|---|---|
| Bitumen Content (%) | 6.0% | |
| Wt. In air (gm) | 1194.4 | 1196.1 |
| Wt. In water (gm) | 698.1 | 704.3 |
| Wt. of saturated surface dry (gm) | 1196.8 | 1198.7 |
| Volume (cm³) | 498.7 | 494.4 |
| Density (gm/cm³) | 2.395 | 2.419 |
| Average Density (gm/cm³) | 2.407 | |
| Dial Reading | 105 | 105 |
| Load (kg) | 1165.0 | 1165.0 |
| Correction factor | 1.04 | 1.09 |
| Corrected Stability@60ºC | 1211.6 | 1269.9 |
| Average Stability@60ºC | 1240.7 | |
| Flow (mm) | 4.0 | 3.9 |
| Average of Flow (mm) | 4.0 | |
have been proven. At present after about one year, no cracks have been observed, which is a better situation than that of the original pavement. After about nine months, hair cracks were observed in the original pavement.

8. CONSIDERATION

(1) Gussasphalt (40)

The Gussasphalt is used for the base course of the orthotropic steel deck pavement. The Gussasphalt, which was developed in Germany and introduced to Japan in 1956, can work as a waterproof layer. The binder of the Gussasphalt contains both petroleum asphalt and the refined natural Trinidad Tobago Lake Asphalt (TLA). The Gussasphalt needs to be cooked and transported to the site at 220°C to 260°C, so that a special cooker is needed. Also because of its fluidity, a special finisher that is different from a conventional finisher is needed. As the Gussasphalt needs special machines, it is sometimes very difficult to adopt the Gussasphalt method in developing countries. The Gussasphalt method is also more costly in developed countries.

(2) SMA

SMA was developed in Germany. As SMA has a high waterproof characteristic, the application of SMA for the base course of the orthotropic steel deck pavement was investigated in the 1990s as shown in Table.4. In May 2001, “Design and Construction Guide of SMA Pavement on Orthotropic Steel Deck” was implemented by Hanshin Expressway Company (5). The pavement of the Suez Canal Bridge was constructed and finished in August 2001. After the application of SMA pavement on the steel decks in Japan, various forms of damage were reported, such as potholes, cracks, rutting, etc (6). After about 2007, almost only the Gussasphalt method was adopted for the steel deck pavement in Japan. After the comparison of various pavement methods for the repavement of the Suez Canal Bridge, the Gussasphalt method was recommended by the study team. In European countries, “Mastic Asphalt Method” is utilized for the orthotropic steel deck pavement. However, this Mastic Asphalt also contains TLA, although the ratio of contents is different from that of Gussasphalt, and needs special machines for the construction. The situation of the construction is almost the same as the case of the Gussasphalt. The SMA pavement of the Suez Canal Bridge cracked in the earlier stage although various countermeasures were adopted to achieve better adhesion to the steel deck and the deflection followability as shown in Sec.2. The countermeasures might have been inadequate for the excessively larger axle weight of more than 25t. After the appearance of cracks, the seepage water might have further deteriorated the adhesive layer and the steel deck. Then the cracks might have further developed. These may be the reasons why this SMA turned out to be unsuitable for this bridge.

(3) Proposed pavement method by GARBLT

This method does not exist in Japan. There is a possibility that this method will be successful for the Suez Canal Bridge. A careful observation of the pavement for the next few years can determine its success. The idea behind this pavement is as follows: First the sand-blasted deck plate is coated with Zinc-Chromate Primer to prevent rust. Over this primer, a thick mixture of tar epoxy and polyurethane is applied. Then the coarse aggregates are scattered to form a resin-bonded surfacing. The mixture works as a bond to fix aggregates and also as a paint to protect the steel deck at the same time. As the surface of the resin-bonded surfacing is only the surface of aggregates, it may be able fix the asphalt pavement above it firmly. The friction between the resin-bonded surfacing and the pavement must be quite large. The adhesion between the resin-bonded surfacing and the steel deck must be large, too. In Photo 40, the worn-out resin-bonded surfacing is shown without peeling off. This may prove that the adhesion between the surfacing and the steel deck is large. As the adhesion between the steel deck and the resin-bonded surfacing is large, and the adhesion between the resin-bonded surfacing and the asphalt pavement is large, the whole pavement system is firmly fixed on the steel deck. This may be the reason why this pavement lasted for a long time despite the traffic load without developing cracks. If this method is proved to be durable and effective, this method will be welcomed by many developing countries, because it is very difficult to repave long span bridges with the Gussasphalt method in these countries. In Japan, coal tar and chromium are hazardous to human health and are already banned (11). If this method is tried in Japan, Zinc-Chrome Primer can be substituted by Inorganic Zinc-Rich Paint for shop painting and Organic Zinc-Rich Paint for the site painting. Tar epoxy can be substituted by the modified tar epoxy. As one good example will not assure future effectiveness, more investigations and trial tests may be needed.

(4) Steel deck thickness

From the specifications for highway bridges in Part 2 Steel Bridges, April 2012, Japan Road Association, the minimum steel deck thickness has been changed from 12mm to 16mm to achieve higher fatigue strength. The Suez Canal Bridge was constructed with 12mm deck thickness based on the older specifications. At the time of the investigation,
no fatigue cracks were found but careful and continuous monitoring is indispensable. At present, a strict axle weight restriction of 13t is enforced so that this could assure better condition of the bridge.

9. CONCLUSION

It was confirmed that if the pavement cracks occurred on the orthotropic steel deck plate, seepage water remained under the pavement and directly above the deck plate, even under the subtropical desert climate in Egypt. This water may have deteriorated the pavement and the steel deck surfaces. The thickness reduction of the steel deck plate was not large at the time of the study.

The condition of the girder surfaces were quite good after about 15 years because of the dry climate.

As technologies develop, some technologies become outdated while some remain valid. The long span bridges are constructed with the best knowledge of the time and the construction cost is high. Therefore, the long span bridges must be properly maintained to secure the traffic flow, even if some of their technologies become outdated. The SMA pavement of the Suez Canal Bridge was removed and the bridge was repaved by the own method of GARBLT, which could be successful but needs to be monitored for the next few years to confirm its effectiveness. The 12mm-thick deck plate needs to be inspected carefully and if fatigue cracks are found, the deck plate needs to be repaired by fatigue crack specialists.

ACKNOWLEDGMENTS: The authors would like to express their sincere thanks to the members of JICA Egypt office, especially Kei Ikegami and Ashraf M. El-Abd and the members of GARBLT, Hala Sayed Helmy, Aly Elsafty Abdalla, and Desoky Osman Desoky, for their support during the study. They are also greatly indebted to Masaru Terada of the Public Works Research Institute of Japan for his advice during the study.

REFERENCES
1) http://www.ismailia.climatemps.com
2) Sugawara, N., Iwahashi, Y., Sakamoto, Y. and Tatsuaki, Y. : Construction of orthotropic steel deck pavement of Suez canal bridge, *Hoso*, Vol.37, No.1, pp. 10-15, Jan. 2002.
3) Design & Construction Manual for Asphalt Pavement, Japan Road Association, Feb. 2006.
4) Hisari, Y. and Tona, M.: Consideration on pavement damages of Hanshin Expressway, *Hoso*, Vol.42, No.9, pp.8-13, Sept. 2007.
5) Defects Inspection Report, Project for Construction of the Suez Canal Bridge, Pacific International Consultant, Chodai, Nov. 2003.
6) Uchida, K., Nishizawa, T., Himeno, K. and Nomura, K.: Study on longitudinal surface crack in pavement on steel plate deck, *Journal of Pavement Engineering*, JSCE, Vol.4, Dec. 1999.
7) Road Bridge Deck Waterproof Manual, Japan Road Association, pp.35, Mar. 2007.
8) Cooperative research on durability of the steel members on bridges - Survey on the inspection method of the orthotropic steel decks, TECHNICAL NOTES of the National Institute for Land and Infrastructure Management, No.471, National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure, Transport and Tourism, Japan Association of Steel Bridge Construction, Aug. 2008.
9) Shimada, T., Onodera, H. and Nishimura, K.: Investigation of pavement cracks of the Hakucho bridge and its repair plan, 59th Technical Research Presentation Conference, Bureau of Hokkaido Development, Ministry of Land, Infrastructure, Transport and Tourism, 2015. https://www.hkd.mlit.gov.jp/ky/giijyutu/splaat0000003ei7-att/IK-5.pdf
10) Tada, H.: Design and Construction of Orthotropic Steel Deck Pavement, Kajima Publisher, 1990.
11) Painting Guide and Manual on Machines, Ministry of Land, Infrastructure, Transport and Tourism, pp.1, Apr. 2010.

(Received January 16, 2017)