Study of Long Span Bridge Design Based on Long Term Maintenance in Developing Countries

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Study of Long Span Bridge Design Based on Long Term Maintenance in Developing Countries

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The author would like to express his sincere thanks to Professor Sugiura, late Professor Shirato, Professor Kawano and Professor Yagi for their advice and encouragement during the course of this study. He had felt very sorry to know the death of Professor Shirato.

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In the beginning, the author thought that if there had been three papers with peer review, then that would have satisfied the criteria for the doctorate thesis. But Professor Sugiura pointed out that if some paper was utilized already by some other author for his doctor thesis, then that paper could not be utilized again for another author's doctorate thesis because of the copyright issues. This surprised the author because two of them had been already utilized and the necessity to submit two more papers had arisen.

The author had been thinking that he would like to try Ph.D degree someday as his elder brother had gotten the degree long ago, but time flies too fast and it becomes so late. When the author was planning to try Ph.D, he was working for Nippon Engineering Company. If he had continued to work for the same company, he might have been able to engage in the Matadi Project until the end and to write a paper about the installation of the dry air injection system. But as he grew older and getting closer to the age of 60, the company threatened him to reduce the salary to 70% as he was not a parachuted government official, which he hated and changed the job to Chodai Company, where the same salary was assured. After his decision, the chairman, Dr. Kawakami, of Nippon Engineering Company tried hard to hold the author back even proposing 1.2 times of former salary. The author was really grateful for Dr. Kawakami's proposal but it was too late. Then why the company threatened to reduce the salary in the beginning was beyond the comprehension of the author.

After beginning to work in Chodai Company, luckily enough, he had a chance to engage in the pavement maintenance work of Suez Canal Bridge in Egypt, which was very interesting and after the project at the site he submitted a paper to JSCE with the permission from JICA and the paper was accepted which is utilized in Chapter 5. When he moved to Chodai, his elder daughter wanted to study abroad and began her study at the Fashion Institute of Technology (FIT), one branch of New York State University. Calvin Klein and Michael Kors are among the graduates of FIT. The tuition fees were not extremely high as FIT was a state university but the living expenses, US$1800/month, was quite high and this influenced a lot on his family finances. But maybe heaven helps those help themselves? At the age of 60, a part of pension began to be supplied to him and he decided to enroll in the doctor course. After about 30 years, to be a student again was a strange feeling but at the same time, it was encouraging to see other doctor students.

This personal doctorate project was secretly proceeding from the point of view of the author as the acquisition of doctor degree was not certain. But his daughters openly told this fact to their friends and in return they seemed to praise the author, which was a little embarrassing and at the same time, funny.

In Japan, it is sometimes said that the doctorate degree is something like a grain of cooked rice under your foot. If you don't take it, it will irritate you. But even if you take it out, you cannot eat it as it is already dirty. Or you cannot live with it, because the degree may not support your life sufficiently.

But in the consultant engineers selection, the doctorate degree is highly appreciated by the client and if you have a doctor degree, it is highly possible that you may win the selection and may be appointed by the client.

This personal doctor project worried financially the author's family but still they encouraged the author. The author must express his sincere thanks to the family members.

Finally the author would like to express his sincere thanks again to Professor Sugiura for his warm encouragement and guidance.

ABSTRACT

For the design of long span bridges in developing countries, in addition to the conventional design method which is developed and utilized in developed countries, there are other design ideas which are specifically needed in developing countries. Various maintenance problems in Japan, which can be utilized for the future design, are reviewed first. Then three maintenance problems in developing countries are selected, they are the problems of the maintenance facilities, the main cable corrosions and the orthotropic steel deck pavement.

From the investigation of the maintenance facilities of the Matadi Bridge, the importance of adoption of manually driven maintenance vehicles, which are easily maintained and whose parts are difficult to be stolen, is found. The difficulty and the importance of maintenance of various trucks, cranes, etc. are found. If the bridge with slightly wider girder width had been constructed, the maintenance work of cables by the cable inspection vehicles would have been easier. But if a slightly wider girder is adopted, the whole structure of the bridge needs to be widened accordingly and the initial construction cost becomes higher. But from the view point of maintenance the wider girder is recommended.

From the investigation of the main cables of the Matadi Bridge, some corrosions were found. But the progress of deterioration is slower than other suspension bridges in Japan, probably due to the less number of occurrence of dew condensation at the site. At two cable bands, the water retention and consequent corrosions of cable wires were found, this might be because OEBK (Organization pour L'Equipment de Banana et Kinshasa) had painted and closed the cable band bottom openings. As the main cables were rusty, it was judged necessary to take some countermeasures. After the investigation, the dry air injection system was adopted. But from the well-drained center cable band, no red rust of wires was found after about 30 years.

When the SMA pavement of the Suez Canal Bridge was investigated, 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 SMA pavement of the Suez Canal Bridge was removed and the bridge was repaved by the proposed method of the General Authority for Roads, Bridges and Land Transport (GARBLT), which could be successful. If this method is durable and effective, this method can be utilized in developing countries, easily.

From the above investigations of three problems, the following feedbacks for the design of long span bridges in developing countries are proposed. The adoption of manually driven inspection vehicles with the redundant driving gears is recommended to ensure the longer service life. But if the longitudinal gradient of the bridge exceeds more than 1%, inspection vehicles need to be electrified. In this case, various countermeasures to avoid the theft are needed. From the observation results of the Matadi Bridge that inside of the well-drained center cable band, no red rust of wires was found, the steel cover corrosion protection system is proposed. It should be easy to maintain this system as this system does not need electrified machines. For the orthotropic steel deck pavement, the method proposed by GARBLT is recommended as this method does not need any special machines. But some improvement is needed to replace the tar epoxy and Zinc-Chrome Primer by some other safe painting material, such as the Zinc rich paint and the modified epoxy.

Chapter 1 INTRODUCTION

1.1 BACKGROUND

There are three important factors for the maintenance of long span bridges in developing countries. One is the acquisition of the fund for the maintenance activities, such as the daily maintenance works, repair works, repainting works, etc. Another is the human resources. To maintain long span bridges continuously, the long term arrangement of well-trained engineers who know well about their bridge is indispensable. The last is the technical knowledge of the maintenance engineers, about the bridge structures, the method of maintenance works, etc. In this study, this technical knowledge for the maintenance of long span bridges is studied.

There are two types of long span bridges. One is the steel structure and the other is the concrete structure. In this thesis only steel structure long span bridges are focused.

In old days, the long span bridges were constructed only in developed countries and those bridges were maintained by engineers who actually planned and designed. Those engineers understood the structural system of long span bridges well and coped with the maintenance problems of those bridges. The knowledge acquired through the maintenance works were then fed back to the design of new long span bridges. In recent years, a large number of long span bridges were built or are being built in developing countries. The maintenance of these bridges is slightly different from the maintenance of those bridges in developed countries. Because the long span bridges are built by the engineers who had experienced in developed countries, the maintenance engineers of developing countries have less knowledge about the structure of the bridges. Even if those maintenance engineers have enough knowledge, the availability of various construction machines, repair tools, spare parts, etc. is limited so that the maintenance works become more difficult. To cope with this situation, some design aspects of long span bridges need to be improved. This is the primary objective in this thesis.

In this chapter, the long span bridges, their histories and maintenance works are briefly introduced first. Then the purpose of this study and the structure of this thesis are explained.

1.2 INTRODUCTION OF LONG SPAN BRIDGES[1)2)]

There are two types of long span bridge. They are a suspension bridge and a cable-stayed bridge. At present, suspension bridges have the longest center spans which approach close to 2000m. The special features of suspension bridges are the cables and the stiffening girders. The stiffening girder supports the live loads such as the traffic load or the train load and transmits the loads to the cables through the hangers. The cables accept the loads from the stiffening girders and transmit the force to the anchorages. The stiffening girder cannot stand by itself and it is totally hung from the cables. Therefore the structure of suspension bridges is more flexible than the conventional short and medium span bridges and produces larger deformations due to the winds or the earthquakes. Therefore the wind resistance design is very important. As an example of suspension bridge, the drawing of the Akashi Kaikyo Bridge is shown in Fig.1.1.

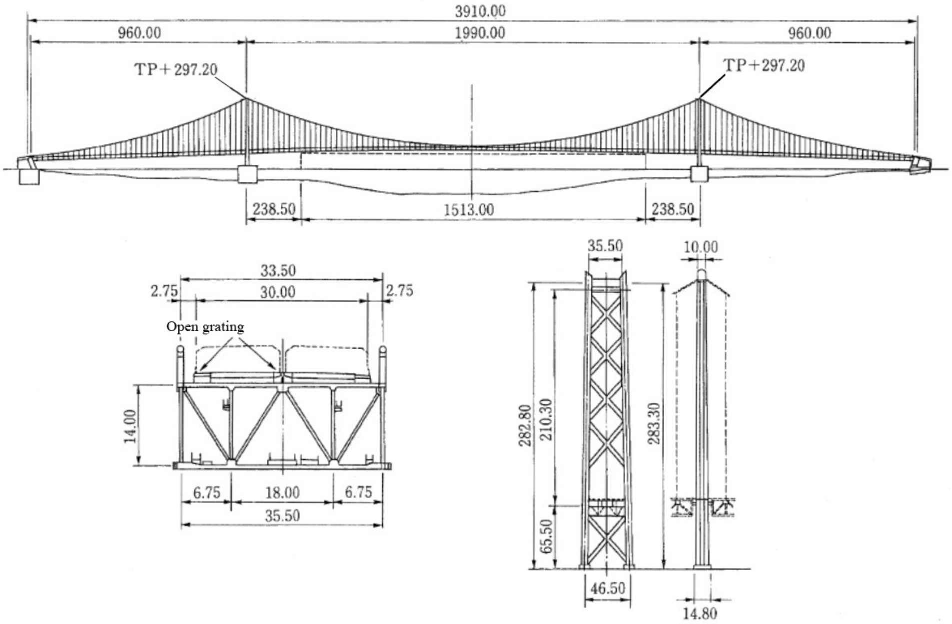


Fig.1.1 Akashi Kaikyo Bridge[1)]

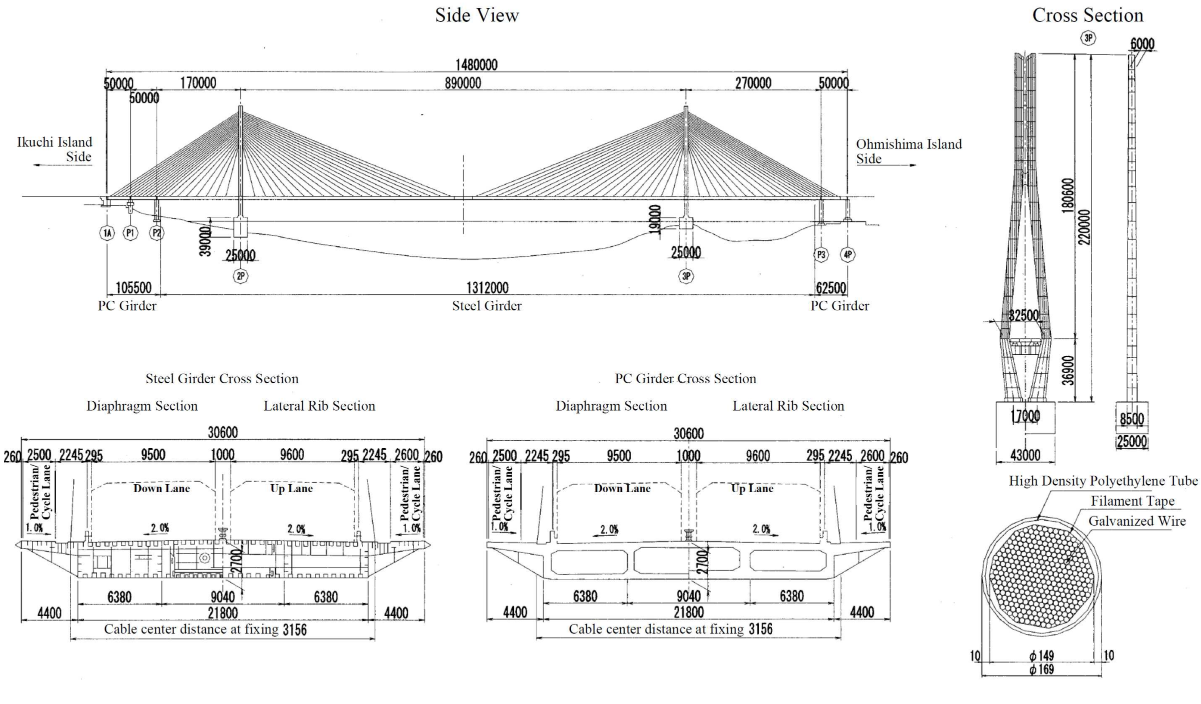


Fig.1.2 Tatara Bridges[2)]

The longest spans of cable-stayed bridges surpassed the length of 1000m, at present. For cable-stayed bridges, inclined cables fixed at the towers pull up the girders. Therefore the bending moments and the axial forces occur inside of towers and girders. The design flexibility of this bridge is quite large. Various cable arrangements and various shapes of towers can be adopted. For longer spans, this bridge type can be adopted and the flexibility of pier arrangement is high. The curved girders also can be adopted and this is impossible for suspension bridges. As longer spans can be adopted, the girders become more flexible than the conventional short and medium span bridges, hence the earthquake design and the wind resistance design become more important. The tensile strength of cables is much higher than that of the structural steel, the economic design of bridges can be achieved. For example, if a 500m span bridge is planned as a steel truss bridge and as a steel cable-stayed bridge, the steel cable-stayed bridge alternative would be much cheaper. The girders of cable-stayed bridges are supported by inclined cables from the towers, the compression force occurs inside of the girders, especially the higher compression force occurs near the towers. Therefore the repair of girders is sometimes difficult as the compression force need to be bypassed by some countermeasures, on the contrary to the girders of suspension bridges which have theoretically no stress inside of the girders. For the longer span cable-stayed bridges, the multi cable fan configuration with two planes is almost always adopted. Two planes are adopted because the torsional rigidity of the girder becomes higher. One of the examples is shown in Fig.1.2, the Tatara Bridge.

1.3 BRIEF HISTORY OF SUSPENSION BRIDGES AND THEIR REHABILITATION[1)]

The brief history of suspension bridges in the world is as follows; the development of modern suspension bridges originated from the invention of air spinning method of steel wires by John Roebling. He designed the Brooklyn Bridge with 487m center span, which adopted the galvanized steel wires. The bridge was completed in 1883. In 1903, the Williamsburg Bridge was completed. The center span of 488m slightly surpassed the center span length of the Brooklyn Bridge. When the Williamsburg Bridge was constructed, it was tried to achieve the cheaper construction cost, the shorter construction period and the longer span than the Brooklyn Bridge, so that the non-galvanized wires, which were cheaper than the galvanized wires were adopted. This adoption caused many maintenance problems of the main cables later on.

In the first half of 20th century, various suspension bridges were constructed and the center span became longer and longer. In 1931, the George Washington Bridge was completed and its 1,066m center span first surpassed 1,000m. In 1937, the Golden Gate Bridge with 1,280m center span was completed. In 1940, the Tacoma Narrows Bridge with the plate girder stiffening girder and the center span of 853m was completed. But 4 months after the completion, the bridge was collapsed by vibrations due to the wind which was comparatively lower than the design wind speed. From this accident, the importance of the dynamic wind resistant design was realized.

In 1969, the Newport Bridge was completed. For this bridge, the PWS (Prefabricated Wire Strand) method was developed and utilized instead of the air spinning method. This method was also developed in Japan and for the suspension bridges of the Honshu-Shikoku Bridge Expressway Company, this PWS method was extensively utilized except for the Shimotsui-Seto Bridge, which were constructed by the air spinning method.

In 1964, the Forth Road Bridge with the 1,006m center span was completed in Scotland near Edinburgh. This was the first bridge outside of USA, which had more than 1,000m center span. In 1966, the 25 de Abril Bridge with the 1,014m center span in Portugal was constructed and completed by the American company.

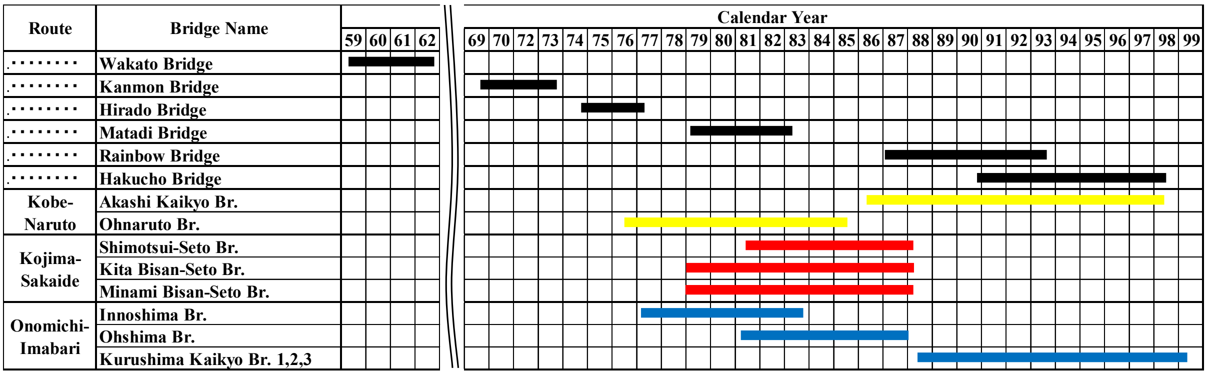
In 1966, a revolutionary suspension bridge, the Severn Bridge with 988m center span in UK, was completed. Instead of American type stiffening truss girders, a box girder was adopted. For hangers, spiral ropes were adopted and the both ends were pinned on the cable bands and on the box girder, instead of American type hangers which are bent over the cable bands and the both end sockets of one cable are fixed on the girder. This might be because the Spiral ropes are stronger than the American type hangers. Also spiral ropes are stiffer so that spiral ropes could not be bent over the cable bands. Because of this change, the cable bands were divided horizontally into upper halves and the lower halves, instead of American type vertical division of cable bands into left halves and right halves. The Severn Bridge adopted the inclined hangers instead of the traditional vertical hangers to achieve the larger rigidity of the whole bridge system[3)]. But after about more than 10 years, hanger breakages happened and finally all of the hangers were replaced by new ones.

In 1973, the Bosphoras Bridge with 1,074m center span was completed. This bridge was designed by the English consultant and employed the same design ideas such as the steel box girder, spiral rope inclined hangers, etc. In 1981, the Humber Bridge with the 1,410m center span, which was the world longest at that time, was completed. This bridge also adopted the steel box girder and the inclined hangers. This bridge adopted the concrete towers. This bridge is known to have the different side span lengths, 280m for North side and 530m for South side. The same design idea was introduced to the Kurushima Kaikyo Bridges in Japan, later on. In 1998, the Great Belt East Bridge with 1,624m center span was completed. This bridge also employed the steel box girder and the concrete towers. But the inclined hangers were not adopted. In recent years, many long span suspension bridges whose center length is more than 1500m were constructed mainly in China. At present Çanakkale 1915 Bridge in Turkey is being constructed[4)]. The center span length is 2,023m. When this bridge will be completed, this bridge will be world longest.

The brief history of suspension bridges in Japan is as follows; the forerunner of the modern suspension bridges in Japan is the Wakato Bridge with 370m center span completed in 1962. For this bridge, spiral ropes were bundled to form the main cables. In 1973, the Kanmon Bridge whose span length was almost twice as long as that of the Wakato Bridge was completed. The center span length is 712m. To construct the main cables of this bridge, PWS method was utilized. One strand consists of 91 wires. For the later suspension bridges, i.e. the suspension bridges of the Honshu-Shikoku Bridge Expressway Company, PWS strands which consist of 127 wires were adopted to achieve the higher effectiveness of construction. In 1977, the Hirado Bridge with 465m center span was completed. The main cables were constructed by the air spinning method. In 1975, the construction of the three routs of the Honshu-Shikoku Bridge Project began. As the first bridge of this project, the Innoshima Bridge with 770m center span was completed in 1983. In 1985, the Ohnaruto Bridge with 876m center span was completed. This bridge employs the double deck truss girder. The upper deck is for the road traffic and the lower deck for the two tracks of the Shinkansen trains in the future. But only one track of the train loading at a time is considered because the trains can be controlled by signals. The same design idea was adopted for the suspension bridges of the Seto Ohashi Bridges. Although four train tracks can be accommodated in the future, only two train loading is considered at a time. During the construction of the Innoshima Bridge and the Ohnaruto Bridge, the Matadi Bridge in the Democratic Republic of Congo (DRC) was also being constructed and completed in 1983. This bridge was constructed by the Japanese ODA project and by the Japanese company, therefore this bridge adopted the Japanese technology. This bridge also has a double deck truss girder with the road traffic on the upper deck and one train track on the lower deck in the future. In January

1988, the Ohshima Bridge with a 560m center span and the first steel box girder suspension bridge in Japan was completed. In April 1988, the Seto Ohashi Bridge was completed and opened to the traffic. This route contains three suspension bridges, i.e. the Shimotsui-Seto Bridge with 940m center span, the Kita Bisan-Seto Bridge with 990m center span and the Minami Bisan-Seto Bridge with 1,100m center span. These bridges have double deck truss girders and serve for both trains and road traffic. The Kita Bisan-Seto Bridge and the Minami Bisan-Seto Bridge shares one anchorage between them. The same design idea to share one anchorage together was later adopted for the Kurushima Kaikyo Bridges which consist of three successive suspension bridges. After the completion of the Seto Ohashi Bridge, in 1998, the Akashi Kaikyo Bridge with the world longest center span of 1,991m was completed. In 1999, the Kurushima Kaikyo Bridges were completed. The First Kurushima Kaikyo Bridge has a 600m center span, the Second Kurushima Kaikyo Bridge has a 1,020m center span and the Third Kurushima Kaikyo Bridge has a 1,030m center span. During this period, except for the Hoshu-Shikoku Bridge Project, two suspension bridges were constructed. In 1993, the Rainbow Bridge with a 570m center span was completed. This bridge has a steel truss girder and serves for both the road traffic on the upper deck and the trains on the lower deck. In 1998, the Hakucho Bridge with a 720m center span was completed. This bridge adopted the steel box girder. The schedule of the construction of major suspension bridges in Japan is shown in Table 1.1. In this Table, the schedule of the Matadi Bridge is also included as a reference.

Table 1.1 Construction of Suspension Bridges in Japan



The suspension bridges of the Honshu-Shikoku Bridge Expressway Company can be divided into two groups. One is the former group whose bridges were constructed before 1988 and these bridges are the Innoshima Bridge, the Ohnaruto Bridge, the Ohshima Bridge, the Shimotsui-Seto Bridge, the Kita Bisan-Seto Bridge and the Minami Bisan-Seto Bridge. The other is the latter group and these bridges are the Akashi Kaikyo Bridge and the Kurushima Kaikyo Bridges. For the former group, the American technologies were mainly adopted, such as the stiffening truss girders, hanger structures, etc. For the Ohshima Bridge, the box girder was adopted for the first time in Japan, which is similar to the box girder of the Severn Bridge. But the hanger structures still adopted the American type hanger structures. For the latter group, Prefabricated Parallel Wire Strand cables covered with High Density Polyethylene Tube (PWS cables, hereinafter) are adopted for the hangers instead of spiral rope hangers of the Severn Bridge. This is because the PWS cables have a higher corrosion resistance. For the Kurushima Kaikyo Bridges, the box girders and the horizontally divided cable bands, the PWS hangers pinned on cable bands and on the girders are adopted which was learned from the

English suspension bridges.

Some of the major maintenance problems of suspension bridges are as follows; the main cable corrosions of the Williamsburg Bridge, the Forth Road Bridge, the Wakato Bridge, the bridges of the Honshu-Shikoku Bridge Expressway Company, etc. the hanger vibration problems of the Severn Bridge, the Bosphoras Bridge, the Akashi Kaikyo Bridge, etc. The Williamsburg Bridge adopted the non-galvanized wires and after 7 years of completion, in 1910, the corrosions of main cables and wire breakages were confirmed by the inspection. Afterwards several times of repair works were carried out, still the corrosion proceeded and the reconstruction of the whole bridge was also investigated. From 1987 to 1988, the cable condition were precisely investigated and it was decided to repair the bridge so that the bridge could be utilized safely for the next 100 year. Some of the repair works were, to connect broken wires, to exchange two strands by new ones, to fill the void of the cable by linseed oil to stop the further deterioration of wires. The main cables of the Forth Road Bridge was opened and inspected in 2004, 40 years after the completion in 1964. Fairly extensive corrosion and wire breaks were found as shown in Photo 1.1[5)]. As the countermeasure against the corrosion, the dry air injection system was introduced in 2009.[4)]

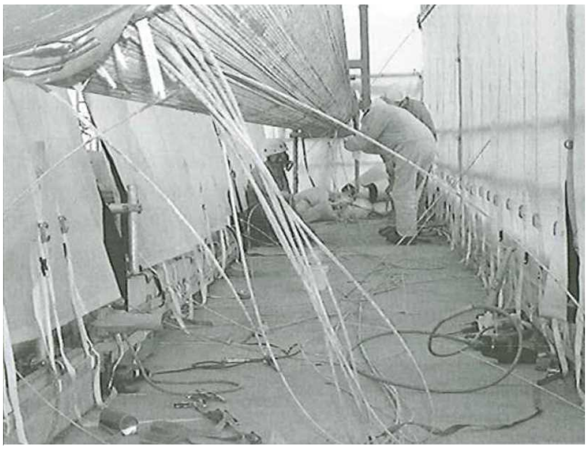


Photo1.1 Wire breakage of Forth Road Bridge[5)]

The main cables of the Wakato Bridge was inspected in 2012[6)]. The main cables of this bridge consist of spiral rope strands. Some wire breakages were found inside of three cable bands but the number of wire breakages was 1 to 51 out of 7351 total wires so that the cables were judged healthy, 50 years after the completion of the bridge. The main cables of the Innoshima Bridge, one of the suspension bridges of the Honshu-Shikoku Bridge Expressway Company, was opened and inspected in 1990, 7 years after the completion of the bridge[7)]. Some corrosions and the water were confirmed inside of the main cables. Afterwards various corrosion prevention countermeasures were investigated and finally the dry air injection system was invented and installed. For all of the former group of suspension bridges of the Honshu-Shikoku Bridge Expressway Company, the dry air injection system was installed before 2002. For the latter group, the dry air injection system was planned from the beginning. The dry air injection system is installed on bridges not only in Japan but also in China and European countries, such as the Rainbow Bridge, the Hakucho Bridge, the Hirado Bridge, the Forth Road Bridge, the Severn Bridge, etc.[8)] The Severn Bridge adopted the inclined spiral rope hangers. As the cross section of the spiral rope is circular, the vibration problem happened[9)].

Photo 1.2 Dampers of
Hanger of Severn Bridge[10)]

To suppress the vibration, dampers were installed as shown in Photo 1.2. As the same type hangers were adopted for the Bosphoras Bridge, the hanger vibration occurred and one hanger plate on the girder close to the tower, which fixed the longest hanger was broken and severed in 2003 due to the progress of the fatigue crack[11)]. The Akashi Kaikyo Bridge and the Kurushima Kaikyo Bridges adopted PWS cables covered with High Density Polyethylene Tube which has a circular cross section, hence the hanger vibration problems happened. Except for the hanger vibration problems, the steel towers of the Severn Bridge were also strengthened and the orthotropic steel deck girders were repaired[10)]. The inclined hangers of the Bosphoras Bridge were exchanged to the vertical hangers in 2015 and the dry air injection system for the main cables was introduced in 2016 because some wire breakages were found near one tower[12)].

1.4 BRIEF HISTORY OF CABLE-STAYED BRIDGES AND THEIR REHABILITATION[2)]

The brief history of cable-stayed bridges is as follows; the history of modern cable-stayed bridges began from the Strömsund Bridge, 183m center span, in Sweden completed in 1955. For cables of this bridge, rocked coil ropes were adopted. Afterwards several cable stayed bridges were constructed by the German technologies. In 1967, the Friedrich-Ebert Bridge with 280m center span was completed. This bridge adopted the multi cable configuration. If a fewer number of cables are adopted for a longer span cable-stayed bridge, the tension of the cables becomes too large and consequently cable fixing structures become too large and complicated. To solve this problem, the multi cable configuration was invented. Afterwards, the longer span bridges always adopt this configuration. In 1975, the St. Nazaire Bridge with 404m center span in France was completed. This bridge first surpassed the span length of 400m. In 1976, the Rokko Bridge with 220m center span in Japan was completed. The wake galloping phenomenon of the two parallel cables was observed for this bridge and to suppress this vibration, cables were tied together by ropes. In 1972, the Kurt-Schumacher Bridge in Germany was completed. The PC box girder for the side span and the steel girder for the center span were adopted for this bridge. The composite structure was adopted to balance the weight of the side span and the center span. The same design idea of the composite girder was adopted for the Ikuchi Bridge and the Tatara Bridge in Japan, later on. In 1970's, one of the characteristics of the cable-stayed bridges in Japan was the adoption of the Parallel Wire

Strand cables (PWS cables, hereinafter.). The PWS cable has the merit of higher tensile strength and the higher modulus of elasticity compared to the locked coil ropes or the spiral ropes. Hence PWS cables are extensively utilized in Japan. In 1980's, various long span cable-stayed bridges were constructed in the world. In Japan, the Meiko Nishi Bridge with 405m center span, the Hitsuishijima Bridge and the Iwakurojima Bridge with 420m center span respectively and the Yokohama Bay Bridge with 460m center span were completed. Also the Luling Bridge with 373m center span in USA, the Alex Fraser Bridge with 465m in Canada, the Rama IX Bridge with 450m center span in Thailand, etc. were completed. The prefabricated parallel wire strand sheathed in a High Density Polyethylene tube (HDPE tube, hereinafter) was developed in Japan. 7mm diameter wires are bundled and covered with a HDPE tube to form a strand at a factory. This PWS cable did not need any anti-corrosion treatment at the site. This strand was utilized for the Yokohama Bay Bridge completed in 1989. This strand is utilized for many bridges in Japan. But as the strands are prefabricated at the factory, comparatively larger machines are needed to construct these cables at the site. In 1990's, the center spans became longer and longer. In 1995, the Normandie Bridge with 856m center span in France was completed and surpassed the formerly longest Yangpu Bridge with 602m center span in China by more than 250m. In 1999, the Tatara Bridge with 890m center span was completed and became the world longest bridge. To suppress the rain vibration of the cables, the indented surface cables were adopted for this bridge. In 2008, the Sutong Yangtze River Bridge with 1,088m center span in China was completed. In 2009, the Stonecutters Bridge with 1,018m center span in Hong Kong was completed. These two bridges employed the indented PWS cables. In 2012, the Russky Bridge with 1,104m center span was completed and this bridge has the world longest span at present[13)]. In recent years, strand cables consisting of seven-wire prestressing strands, individually greased and sheathed with Polyethylene, bundled and encased in a PE sheathing tube at the construction site are often adopted for the cable-stayed bridges, especially in Europe. Partly because these cables do not need larger construction machines. The Russky Bridge adopted these cables, too.

As the girders of the cable-stayed bridges are supported by the cables, the maintenance works of the cables are needed in addition to the normal maintenance works of the conventional steel girder bridges. Generally speaking, cable-stayed bridges have longer spans, hence the displacement due to the live loading is larger. Then the maintenance of bearings or expansion joints become more important. Some of the maintenance problems are as follows; rusty wires of cables due to the deterioration of cable covers and consequent water seepage, the breakage of cable dampers, cable cover breakage due to the collision of a container truck and consequent water seepage inside of the cable, water seepage into the cable fixing structures, water seepage into a box girder from a cable fixing structure, fatigue cracks of the orthotropic steel decks, etc. The breakage of the polyethylene tubes of cables of the Hitsuishijima Bridge at the cable bending buffers due to the cable vibrations was found. The same kind of problem is also reported for the Yakohama Bay Bridge. In 2012, the all of the cables of the Luling Bridge in USA were replaced by new cables[14)]. This bridge, completed in 1983, adopted the prefabricated parallel wire strands consisting of 1/4 inch (about 6.4mm) diameter wire and encased in a black HDPE pipe, and the space between is grouted. The damages to the HDPE pipes occurred during the construction and the damages were repaired. Later cracks developed in these HDPE weld repairs. Consequently extensive water leakage and corrosion of wires were found. After the investigation, it was decided to replace all of the cables by the parallel strand cables consisting of seven wire prestressing strands, greased and sheathed with HDPE, bundled and encased in a HDPE sheathing pipe at the site. The corrosion of wires, the PE joint separation and the newly adopted parallel strand cables are shown in Photos 1.3 to 1.5.

Photo 1.3 Corrosion of wires[14)]

Photo1.4 PE pipe joint separation[14)]

Photo 1.5 Newly adopted parallel strand cable[14)]

The Stone Mastic Asphalt pavement was adopted for the original pavement of the orthotropic steel deck pavement of this bridge completed in 1983. In 1994, the failures and delaminations of significant areas were observed. After the experiment of various pavement methods, the bridge deck was repaved by the Steel Fiber Reinforced Concrete (SFRC) in 2016[15)].

1.5 PURPOSE OF THIS STUDY

Suspension bridges and cable-stayed bridges have different features from other conventional short and medium span bridges. Because of these differences, these two types of bridge often suffer different types of damages.

After the opening of these two types of bridges of the Honshu-Shikoku Bridge Expressway Company, many maintenance problems were reported. The causes of these problems were analyzed and the defects were repaired to restore the bridges to the original healthy condition. The same situation happens for the long span bridges in developing countries, constructed by the Japanese ODA. For the maintenance of these long span bridges, the circumstances are slightly different from Japan and a slightly different counter measures which take into account these different circumstances are needed.

For the long span bridges of the Honshu-Shikoku Bridge Expressway Company, the major problems are as follows; the corrosion protection of suspension bridge main cables, corrosion of hangers, vibration problems of hangers, noise problems of expansion joints, noise problems of the end links, vibration problems and the damage to the PE covers of the cable-stayed bridge cables, the maintenance problems of inspection vehicles, etc. In this study, the main cable condition and the condition of inspection vehicles of the Matadi Bridge are investigated. Then the condition of the orthotropic steel deck pavement of the Suez Canal Bridge is also investigated. These maintenance activities and the improvement of the problems of long span bridges are investigated and analyzed so that the analysis results can be utilized for the future design of long span bridges in developing countries. This is the purpose of this study. The explanation above can be summarized in Table 1.2 below. The fatigue and fatigue cracks of the orthotropic steel decks form really a large problem but these problems are already extensively investigated and the results are summarized[16)] so that this problem is not discussed in this study.

Table 1.2 Purpose of Study

Kind of Long Span Bridges	Different Features other than conventional shorter span bridges	Purpose of Study
Suspension Bridge	Main cable Hangers Orthotropic Steel Deck and its pavement Expansion joints Inspection Vehicles	The maintenance activities and the improvement of the problems of long span bridges, both in Japan and in developing countries, are investigated and analyzed so that the analysis results can be utilized for the future design of long span bridges in developing countries.
Cable-Stayed Bridge	Cables Orthotropic Steel Deck and its pavement Expansion joints Inspection Vehicles	

1.6 STRUCTURE OF THIS THESIS

The structure of this thesis is shown in Fig.1.3 below.

In Chapter 1, the purpose of study is explained with the introduction of long span bridges, their history and overcoming technical issues. The structure of this thesis is explained.

In Chapter 2, various maintenance problems of the long span bridges of mainly Honshu-Shikoku Bridge Expressway Company and their countermeasures are analyzed and explained. Some of these results can be utilized directly for the repair of the same kind of problems occurred on the long span bridges in developing countries or utilized directly for the design of new bridges in developing countries, but some cannot be applied directly due to the circumferential differences between Japan and developing countries. In Chapter 2,

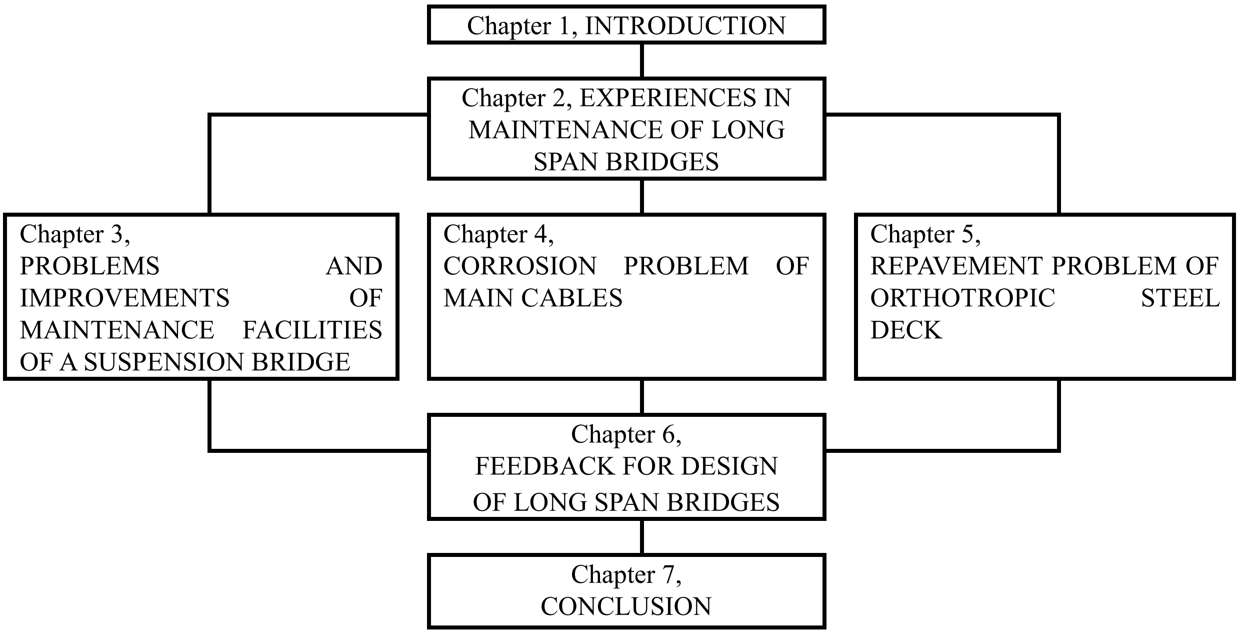


Fig.1.3 Structure of Thesis

11 problems are introduced and the countermeasures are shown. Out of these 11 problems, 3 problems, which need different approach for the solutions in developing countries, are selected and further studied in Chapters 3 to 5.

In Chapter 3, the condition of maintenance facilities of the Matadi Bridge was investigated and the solution plan was proposed based on the analysis of the situation in the developing country.

In Chapter 4, the main cable corrosion problem of the Matadi Bridge was investigated and the condition of corrosion was analyzed. The method of solution was investigated and the solution method was proposed.

In Chapter 5, the condition of the orthotropic steel deck pavement of the Suez Canal Bridge was investigated precisely and the countermeasure of pavement utilizing the Gussasphalt method was proposed. But due to the lack of the special machines needed for the Gussasphalt method, the orthotropic steel deck pavement method of the Egyptian side was adopted. This method is analyzed.

In Chapter 6, based on the analysis of Chapter 3, 4 and 5, the feedback for the design of long span bridges especially in developing countries is proposed.

In Chapter 7, the concluding remarks is given.

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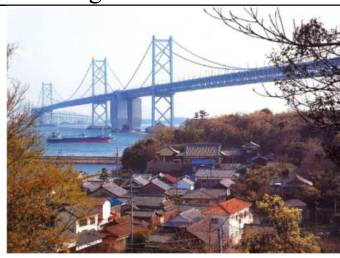
Chapter 2 EXPERIENCES IN MAINTENANCE OF LONG SPAN BRIDGES

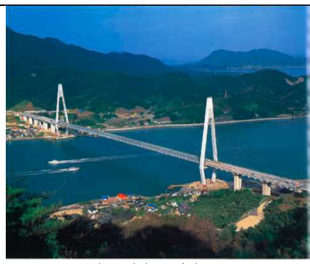
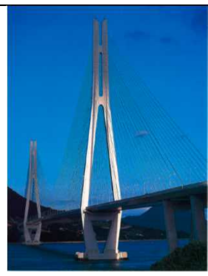
2.1 INTRODUCTION

In this Chapter, various problems encountered during the course of the maintenance works of mainly Honshu-Shikoku Bridge Expressway Company and their countermeasures are introduced. They are as follows; corrosion of main cables of suspension bridges, damages of hangers of suspension bridges, damages to cables of cable-stayed bridges, noise problems of expansion joints, issues of inspection vehicles, etc. The bridges which are listed in Table 2.1, are mainly studied in this chapter.

Some solutions or countermeasures can be adopted directly for the design and the repair of long span bridges in developing countries. Some solutions or countermeasures cannot be adopted directly because of the circumferential differences in developing countries. 11 problems are discussed in this chapter. Based on the experiences of these problems, 3 problems in developing countries are selected for further study. This is because these 3 problems need solutions from a different point of view of developing countries.

Table 2.1 Bridges and their names

<table>
<tr>
<td rowspan="2">Bridges of Honshu-Shikoku Expressway Company[1]</td>
<td>Kobe-Naruto Route</td>
<td colspan="2">
Akashi Kaikyo Bridge,
The span of this bridge is 960m+1991m+960m=3911m. Completed in 1998. This is the world longest suspension bridge at present.</td>
<td colspan="2">
Ohnaruto Bridge
The span of this bridge is 93m+330m+876m+330m=1629m. Completed in 1985. This bridge can accommodate both road traffic and Shinkansen train. The natural condition of this bridge is the severest among Honshu-Shikoku Bridges. Strong tidal current and strong wind.</td>
</tr>
<tr>
<td>Seto Ohashi Bridge</td>
<td>
Shimotsui-Seto Bridge,
The span of this bridge is 230m+940m+230m=1400m. Completed in 1988. This bridge, other suspension bridges and cable stayed bridges of this route have a double deck truss girder with lower deck for trains and upper deck for road traffic. The main cables were constructed by AS(air spinning) method.</td>
<td colspan="2">
Hitsuishijima and Iwakurojima Bridge,
The span of these twin cable-stayed bridges is 185m+420m+185m=790m. Completed in 1988. For cables, PWS cables are adopted.</td>
<td>
Kita(right) and Minami(left) Bisan-Seto Bridge,
These two suspension bridges share one anchorage at the middle together. The spans are, 274m+990m+274m=1538m and 274m+1,100m+274m=1648m respectively. Completed in 1988.</td>
</tr>
</table>

<table>
<tr>
<td rowspan="2"></td>
<td>Onomichi- Imabari Route</td>
<td>
Innoshima Bridge,
The span of this bridge is 250m+770m+250m=1270m. Completed in 1983. This is the first suspension bridge of the Honshu-Shikoku Bridge Expressway Company.</td>
<td colspan="2">
Ikuchi Bridge,
A cable-stayed bridge completed in 1991 with the span arrangement 150m(47.5m+47.5m+55m) +490m+ 150m(55m+47.5m+47.5m) =790m.
The side spans are PC girders. This was the world longest span cable-stayed bridge when completed.</td>
<td>
Tatara Bridge,
A cable-stayed bridge completed in 1999 with the span arrangement 270m+890m+320m=1480m. A part of side spans is PC girder. This was the world longest span cable-stayed bridge when completed.</td>
</tr>
<tr>
<td>Onomichi- Imabari Route</td>
<td colspan="2">
Ohshima Bridge,
A single span suspension bridge with box girder. Completed in 1988. This is the first bridge to adopt a box girder in Japan. Span arrangement is 140m+560m+140m=840m</td>
<td colspan="2">
Kurushima Kaikyo Bridge,
The spans of 1^{st}, 2^{nd} and 3^{rd} bridges are as follows;
50m+140m+600m+170=960m
250m+1020m+245m=1515m
260m+1030m+280m=1570m.
For these bridges, box girders were adopted. For hangers, PWS cables were adopted.</td>
</tr>
</table>

2.2 MAIN CABLES OF SUSPENSION BRIDGES

2.2.1 CONVENTIONAL CORROSION PROTECTION SYSTEM

For the main cables of the suspension bridges of the Honshu-Shikoku Bridge Expressway Company which were constructed earlier, namely, the Innoshima Bridge, the Ohnaruto Bridge, the Ohshima Bridge, the Shimotsui-Seto Bridge, the Kita Bisan-Seto Bridge & the Minami Bisan-Seto Bridge (the former group, which were constructed before 1988.), the corrosion protection system shown in Fig. 2.1 was adopted. For the suspension bridges constructed later, namely the Akashi Kaikyo Bridge and the Kurushima Kaikyo Bridges (the latter group) , different corrosion protection systems were adopted.

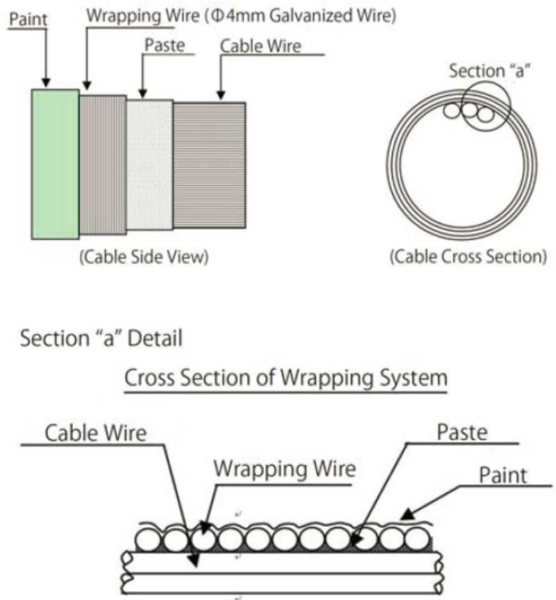


Fig.2.1 Corrosion protection system of Main Cable[2)]

2.2.2 INVESTIGATION OF CABLE CORROSIONS[2)]

To find a better corrosion protection system for the Akashi Kaikyo Bridge, which employs the higher strength wires (160kgf/mm^2→180kgf/mm^2) and a lower safety factor (2.5→2.2), the main cable of the Innoshima Bridge which was completed in 1983, was opened in November 1989 to investigate the condition of corrosion. Some water was confirmed inside of the cable and some corrosion was found on the cable. From this investigation, it became clear that the wrapping paste became deteriorated over time, the paint was cracked due to the thermal elongation/contraction of the cable and the moisture protection was lost. Afterwards, the main cables of the other suspension bridges, namely the Ohnaruto Bridge, the Ohshima Bridge and the Kita Bisan-Seto Bridge, were also investigated. The following facts were confirmed.

(1) Some water was confirmed inside of the cable.

(2) The bottom part and the side parts of the cable were wet but the upper part was almost dry.

(3) The wrapping paste had deteriorated and changed into a water retention body.

(4) The corrosion was confirmed over the whole length of cable.

(5) The corrosion was confirmed along the cable. Up to a few layers of wire from the surface were covered in red-rust.

(6) Inside of cable bands, cables were healthy except for some white-rust, which were confirmed on the lower side of cables.

(7) Corrosion has occurred from an earlier time.

(8) Even if the high performance corrosion protection wrapping paste is used, only surface wires, which are coated with the paste, are protected. Inside wires cannot be protected. (From the inspection results of the Higashi-Ohi Bridge)

2.2.3 CAUSES OF CABLE CORROSIONS[2)]

The causes of cable corrosions are thought to be as follows; From the investigation of existing bridges, some water which may have remained there from the time of the construction or may have come from the surface paint cracks after the completion of the bridge, was confirmed inside of the cables. From this fact, inside of the cable, the process of the evaporation and the condensation depending on the outside temperature may be repeated as illustrated in Fig.2.2. This may have led to the corrosions of wires.

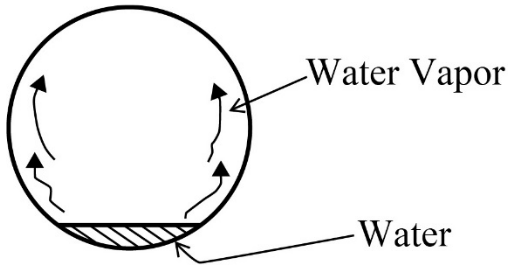


Fig.2.2 Concept of Corrosion Mechanism

The fact that the galvanized wire is not effective against the humidity was already confirmed by the experiment in which the galvanized wire was wrapped by a wet strip of gauze and the wire was rusted in an earlier stage.

For the main cables, not only the surface layer but also a few sub-surface layers from the outside were rusty. The sides of the cables were more rusty and it was thought that the wet condition of the sides continued longer. The fact that the deteriorated wrapping paste, which was changed into a water retention body, also may have accelerated the rusting process.

2.2.4 HUMIDITY INSIDE OF CABLES[2)]

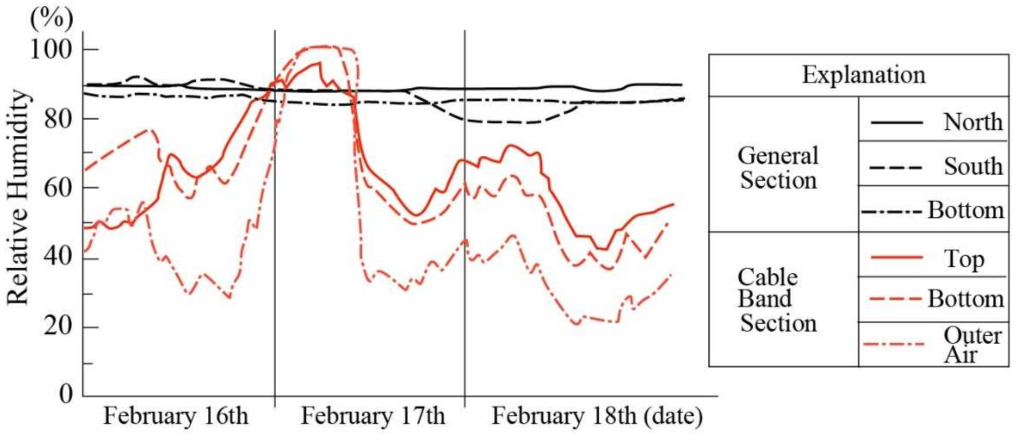


Fig.2.3 Relative Humidity inside of Cable[2)]

The inside condition of the main cables of the Ohnaruto Bridge was investigated. The result is shown in Fig.2.3. The following facts were confirmed.

(1) The humidity of the ordinary section is continuously high without being influenced by the outside air humidity.

(2) The humidity of the cable band section is closely related to the outside air humidity.

2.2.5 DRY AIR INJECTION SYSTEM[2)]

In the beginning, various new wrapping pastes were tested to find the better cable corrosion protection system. But as a result, the following fact became clear.

When water does not exist inside of the cable, no rust occurs.

But when the water exists, the cable becomes corroded irrespective of kind of wrapping pastes.

Then it was decided to develop a dry air injection system of main cables from 1993 as it was practically almost impossible to protect the cables from the water by the conventional cable wrapping system only. Various cable model specimens were tested and the dry air injection systems were tested on the existing suspension bridges. The following facts became clear.

(1) The dry air injection system will dry out the whole main cable and can prevent the corrosion. If the dry air injection system is installed, the wrapping paste is not needed.

(2) The dry air can be injected from the surface of the main cable. The injected dry air will be diffused to the whole sectional area of the cable.

(3) If the relative humidity is less than 60%, the corrosion can be prevented. (The target relative humidity is set at 40%, which is low enough for corrosion prevention.)

(4) From one dry air injection band, the humidity of 200m to 250m section can be suppressed below 40%.

(5) It is very important to secure the sealing of the cable. Cable bands need to be sealed properly. For the former group, the cable surface was coated by the flexible type fluororesin paint before the installation of the dry air injection system.

(6) The completely deteriorated wrapping paste is powdery and has neither the corrosion protection function nor the corrosion acceleration function.

From the facts above, the dry air injection system was introduced to the former group without removing the deteriorated wrapping paste and with the assurance of air tightness of cable bands and the cable surfaces. For the Akashi Kaikyo Bridge, the main cables were first wrapped by the conventional wrapping wires without the wrapping paste and then wrapped by rubber wrapping[3)].

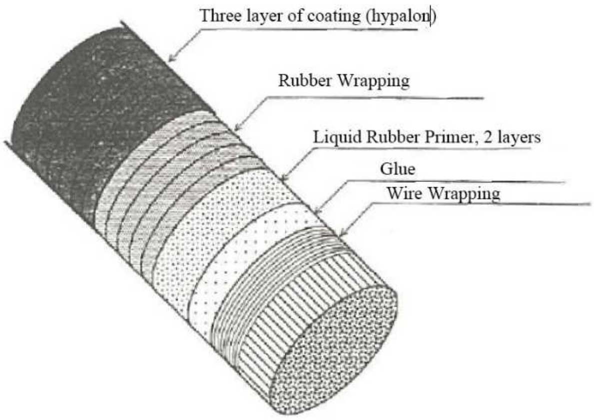


Fig.2.4 Rubber Wrapping of the Akashi Kaikyo Bridge[3)]

For the Kurushima Kaikyo Bridges, the main cables were wrapped by S-shaped wrapping wires which could ensure better sealing.

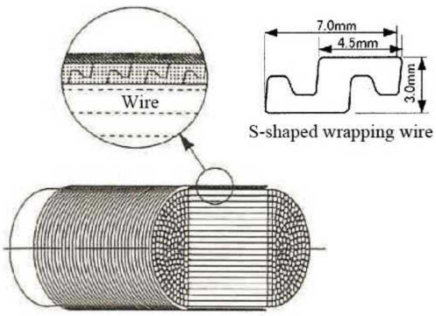


Fig.2.5 S-shaped Wrapping Wires of the Kurushima Kaikyo Bridges[4)]

Until 1999, the installation of the dry air injection system for the three suspension bridges of the Seto Ohashi Bridge had been finished. The length between the dry air injection cover and the exhaust cover is from 228m to 288m as shown in Fig.2.6 below.

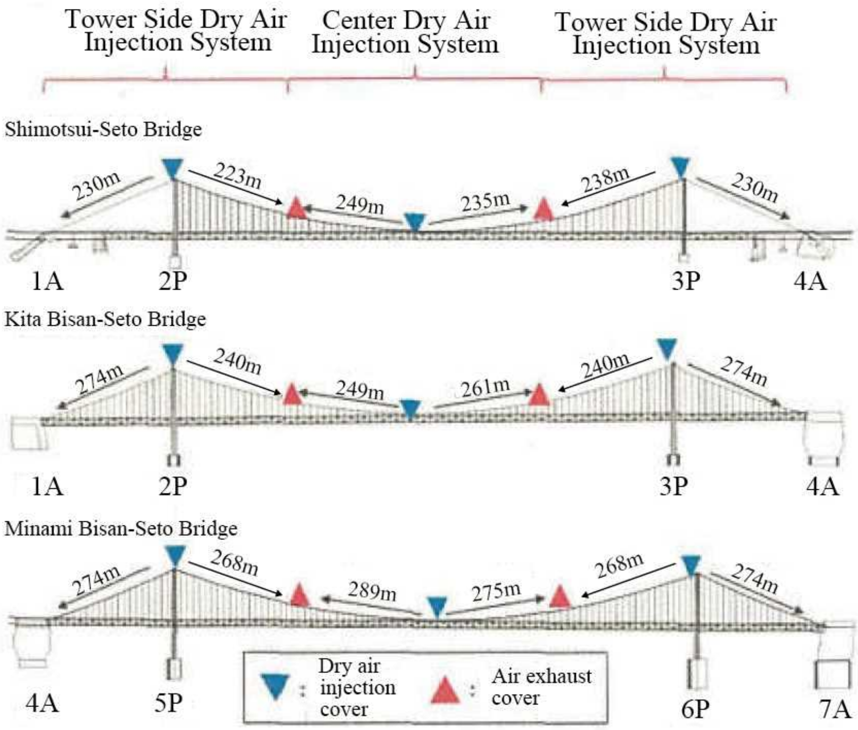


Fig.2.6 Configuration of Dry Air Injection System before Improvement[5)]

With this configuration, the relative humidity near the injection bands became lower but the far ends from the injection bands, i.e. near the exhaust bands, the higher relative humidity remained. The relative humidity sometimes exceeded 60% which was much higher than the 40% target relative humidity. To resolve this problem, it was planned to increase the number of injection bands to reduce the length between the injection bands and the exhaust bands. From 2005 to 2015, this improvement was implemented and the result is shown in Fig.2.7. With this improvement, the relative humidity of the cables became lower than 40%[5)].

Another improvement of the dry air injection system is the addition of the pre-cooler system. For the dry air injection system of the Honshu-Shikoku Bridge Expressway Company, the dry dehumidifier which utilized silica gel in a rotor, was adopted so that the 40% relative humidity air could be acquired even in the winter[6)].

But this system turned out to be slightly inadequate when the temperature was high and the relative humidity was also high in the summer[7)]. To solve this problem, a pre-cooler, a wet dehumidifier which had the same system as the conventional air conditioners, was introduced in front of the existing dehumidifier. With this introduction, the target value of 40% relative humidity was attained and at the same time, the running cost became lower[8)]. Afterwards, this system was introduced to other suspension bridges, as well[5)].

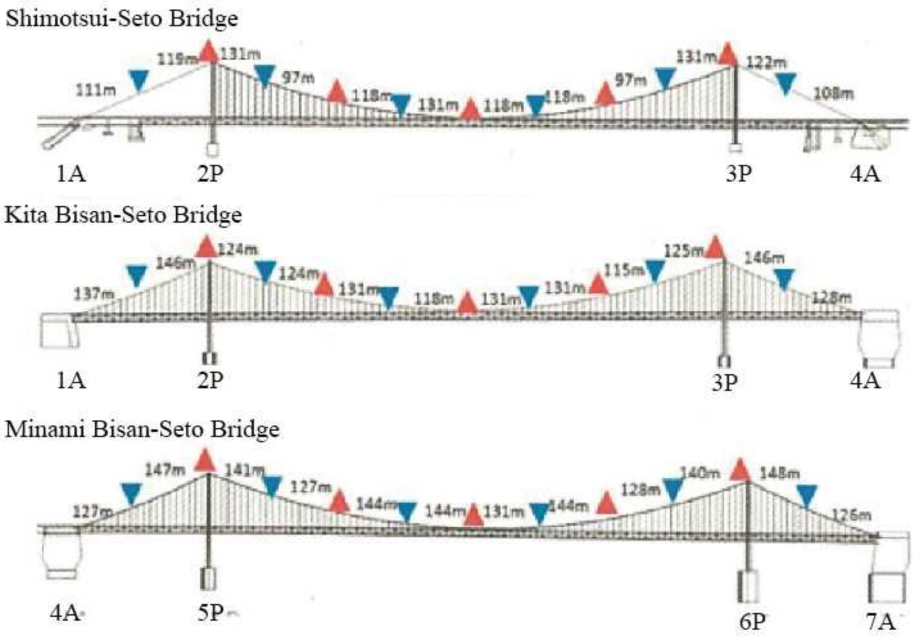
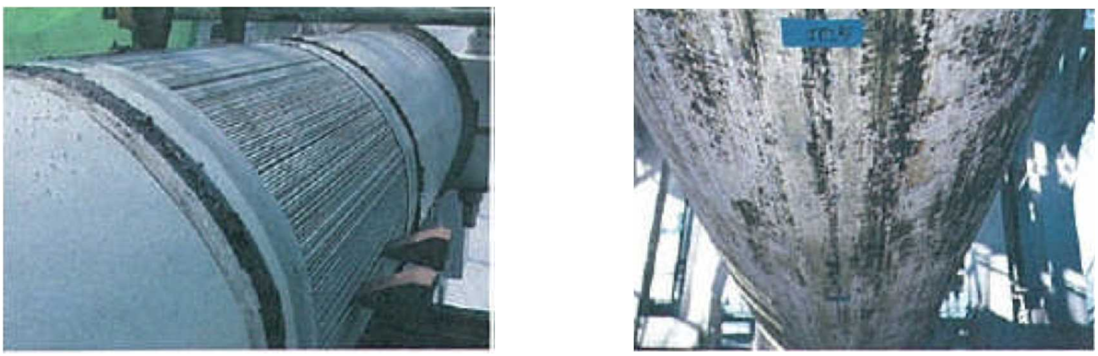


Fig.2.7 Configuration of Dry Air Injection System after Improvement[5)]

The effectiveness of the dry air injection system was examined for the Kurushima Kaikyo Bridges after 8 years from the completion of the bridge[8)]. The result is shown in Photo.2.1 and 2.2. In Photo.2.1, the condition of the First Kurushima Kaikyo Bridge and that of the Kita Bisan Seto Bridge without the dry air injection system and inspected after 8 years from completion, are compared. From this comparison, the effectiveness of the dry air injection system is quite clear.



(a) Unwrapping Inspection of First Kurushima Kaikyo Bridge in 2007, 8 years after completion

(b) Unwrapping Inspection of Kita Bisan-Seto Bridge in 1996, 8 years after completion

Photo.2.1 Comparison of Cables with and without dry air injection system[8)]

In Photo.2.2, the condition of the inside of the main cables are shown. Only some white rust was confirmed on the surface layers of wires. It can be concluded that the cables with the dry air injection system are far healthier than the cables without the dry air injection system.

(a) First Kurushima Kaikyo Bridge

(b) Third Kurushima Kaikyo Bridge

Photo.2.2 Condition of Main Cables of Kurushima Kaikyo Bridges[8)]

2.2.6 DESIGN IDEA AND SOME PROBLEMS

The design idea of the dry air injection system originated from the corrosion protection of cables of the existing bridges which were coated with the wrapping system shown in Fig.2.1. As the wrapping paste which had been already turned into the water retention body could not be removed, the only way to dry out the inside of main cables was the adoption of the dry air injection system. The same design idea was adopted for both the Akashi Kaikyo Bridge and the Kurushima Kaikyo Bridges, although their adopted systems were different. The design idea of the cable wrapping system of the Honshu-Shikoku Bridge Expressway Company can be expressed as the closed system.

The flow of the cable wrapping system development can be summarized as shown in Fig.2.8.

Original cable wrapping system with wrapping paste → Corrosion of wires was found. → Dry air injection system was developed and installed on existing bridges. All of cables were caulked to prevent air leaking. Closed system for outside air and rain. →

Rubber wrapping system with dry air injection system for Akashi Kaikyo Bridge was developed. Closed system for outside rain and air. → S-shaped wrapping wire wrapping system with dry air injection system for Kurushima Kaikyo Bridges was developed. Closed system for outside rain and air.

Fig.2.8 Flow of the cable wrapping system development

As the dry air injection system can dry out the inside of the cable effectively, the corrosion prevention performance is already proved by many application examples. During the operation of the dry air injection system, the humidity of the exhaust air from the exhaust bands on the cables needs to be monitored to confirm the effectiveness of the system. As the dry air blowers are the electrified machines, they may need to be replaced after around ten years. These are the difficult points of the dry air injection systems.

The dry air injection system of the rubber wrapping system of the Akashi Kaikyo Bridge was tested for 3 months from November 1997, before the completion of the bridge[3]. After about 2 weeks of operation, the effect of the dry air injection was confirmed up to 2 to 3 panels section, about 28m to 42m section. But during the operation suspension for about 2 weeks due to the new year holidays, the humidity condition of this section returned closely to the original condition. Therefore the operation suspension, such as due to the electric blackout, should be avoided as far as possible. The secure electricity supply system is indispensable for this system. After about 3 months operation, the effect of the dry air injection was confirmed for 4 panels section, about 56m section. If 150m section needs to be dried by one dry air injection cover, some more time is needed.

Another problem is the deterioration of the caulking materials of the cable bands. From the inspection of the upper surfaces of the cable bands of three suspension bridges of the Seto Ohashi Bridge, air leaking was confirmed. In 2015, the lower side of the cable bands were inspected from the cable inspection vehicles to confirm the air leakages from the caulking materials. The air leakages were confirmed on many cable bands due to the deterioration of the caulking materials. These caulking materials were replaced 14 years ago. The caulking of cable bands of three bridges is being repaired from 2016 as a 6 year project[5].

The introduction of the dry air injection system with the S-shaped wrapping wires for new suspension bridges is well established, as the S-shaped wrapping wires can achieve the higher air tightness. For the existing suspension bridges, the introduction of the dry air injection system by adopting the flexible type paint on the cable surfaces and the caulking of cable bands is well established and may be the only way to stop the progress of cable corrosions. But in both cases, the maintenance of electric machines, the continuous operation, the monitoring of the humidity of air exhaust covers, renewal of deteriorated caulking materials of cable bands, etc. are indispensable.

2.2.7 DIFFICULTY OF MAIN CABLE REPLACEMENT

As stiffening girders of suspension bridges are suspended from main cables, it is very difficult to replace main cables. First new main cables beside or over the existing main cables, need to be constructed. This means that new anchor structures to fix new cables are needed. Then all of the dead weight of the stiffening girder needs to be transferred to the new main cables. Afterwards old main cables can be dismantled and removed.

There are two examples of cable replacement in France[9), 10)]. One is the Tancarville Bridge with 608m center span completed in 1959. The other is the Aquitaine Bridge with 394m center span completed in 1967. For the Tancarville Bridge, two new cables over and side by side of the each old cable were constructed. Afterwards the girder is suspended from the four new main cables and the old main cables were removed in 1999.

Photo 2.3 Tancerville Bridge in France before the cable replacement[11)]

For the Aquitaine Bridge, the stiffening truss girder was widened to receive the new main cables at the side ends of the girder. New main cables were constructed next to and outside of the existing main cables. Afterwards the dead weight of the girder was transferred from the old cables to the new cables in 2002.

Although it is not impossible to replace main cables, it is very difficult and costly therefore it is far more economic to keep main cables healthy for as long as possible.

As the main cables of the Forth Road Bridge were found out to be significantly corroded, a feasibility study to replace main cables was performed[12]. The span of this bridge is 408m+1006m+408m. One of the plans to replace cables is shown in Fig.2.9 and 2.10. In this plan, a new anchorage is proposed. One of the reasons to adopt new anchorage is the easiness to fix cable strands on the anchorage. To fix new cable strands to the old anchorage where there are already existing strands, is a very complicated difficult work.

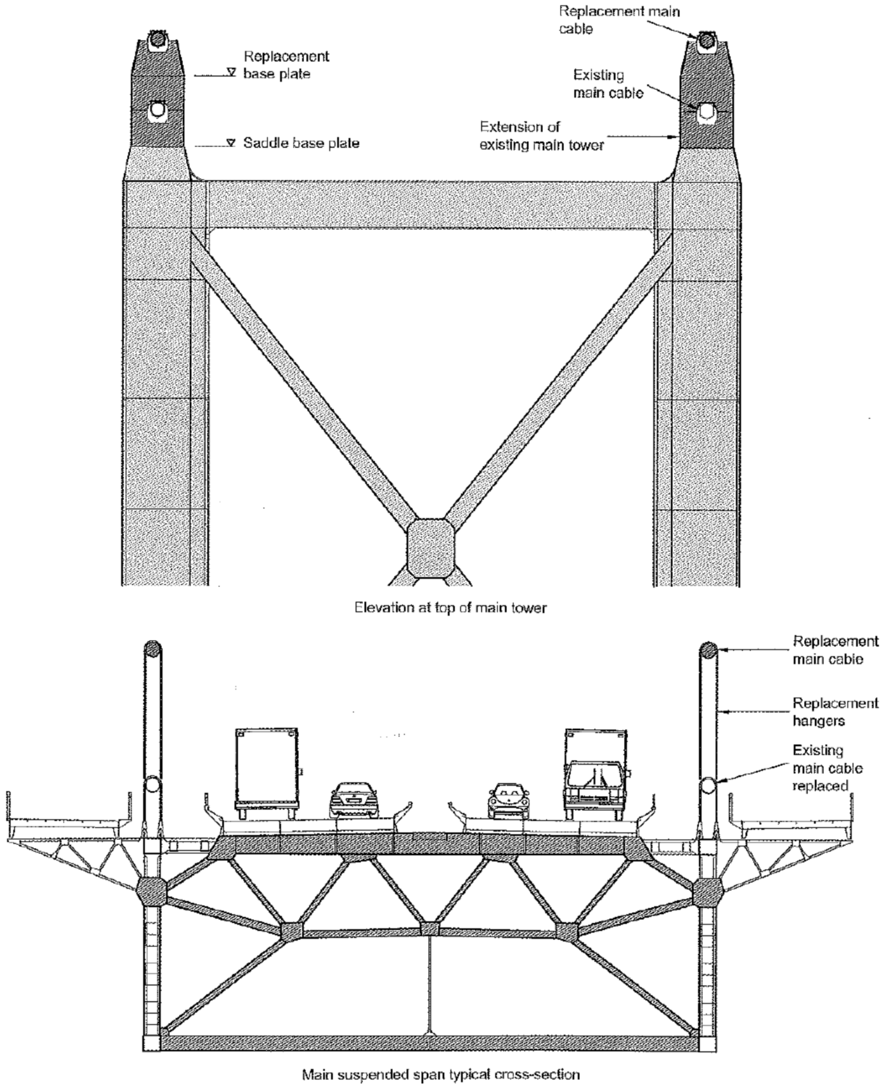


Fig.2.9 Replacement of main cables of Forth Road Bridge[12]

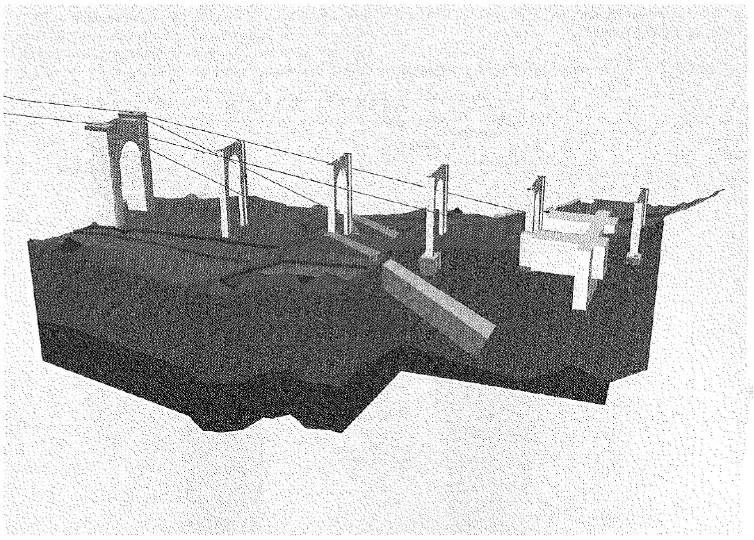


Fig.2.10 New anchorage for Replacement of main cables of Forth Road Bridge[12)]

2.3 HANGERS OF SUSPENSION BRIDGES

2.3.1 AKASHI KAIKYO BRIDGE[13)]

C.F.R.C ropes (Center Fit Rope Core ropes) were adopted for the hangers of the former group of suspension bridges. PWS (Prefabricated Wire Strand) Cables sheathed in Polyethylene (PE) tubes were adopted for the hangers of the latter group of suspension bridges. C.F.R.C ropes as shown in Sec.2.3.2, have a complicated cross section hence no wind induced problems occurred. But C.F.R.C ropes are only painted and are not covered with PE tubes. Therefore some corrosion problems occurred and this case is explained in Sec2.3.2.

Unlike C.F.R.C ropes, PWS cables are covered with PE tubes and no corrosion problems occurred so far. But as the cross section is closely circular therefore some wind induced problems occurred both for the Akashi Kaikyo Bridge and the Kurushima Kaikyo Bridge.

For the Akashi Kaikyo Bridge, two hanger ropes are arranged in parallel on one point of the upper truss chord. The occurrence of the wake galloping, which appears on the downstream hanger due to the disturbance of air flow by the upstream hanger, was feared. But the distance between two hangers is about 9 times the hanger diameter so that it was judged that the galloping would not occur from the precedent examples of double cables. Only the dampers to suppress the vortex-induced vibrations were installed.

After the bridge completion, in July 1998, a typhoon hit the area of the bridge and large amplitude vibrations, which could not be anticipated from the former examples, occurred and some dampers were broken. (Photo2.4) Afterwards it was found by the wind tunnel test utilizing the two dimensional actual size rigid model, that the vibration was the wake flatter which were often observed on the electric power lines. To suppress this vibration, double helical wires with 9mm diameter and the 800mm winding pitch over the 87mm diameter hanger rope were adopted. These helical wires suppressed the vibrations well (Photo 2.5). A few typhoons hit the bridge after the installation of the helical wires but no vibrations were observed and the effect of the helical wires were confirmed.

Photo 2.4 Breakage of damper[13)]

Photo 2.5 Helical Wires around Hangers of Akashi Kaikyo Bridge[13)]

The vortex induced vibrations of the hangers of the Kurushima Kaikyo Bridge were expected during the design stage and it was concluded that it was not necessary to install dampers as the vibration amplitude was not large. But after the opening of the bridge, these dampers shown in Photo 2.6, were installed so as not to impact the road users. These problems were solved by the application of suitable dampers.

Photo 2.6 Damper for Hanger of Kurushima Kaikyo Bridge[13)]

2.3.2 OHNARUTO BRIDGE[14)15)]

The Ohnaruto Bridge is a suspension bridge with the center span of 876m and was completed in June 1985. The natural conditions surrounding this bridge are the severest among Honshu-Shikoku Bridges, with strong winds and strong tidal currents.

Photo 2.7 Ohnaruto Bridge[1)]

The hangers of the Innoshima Bridge, completed in 1983 were inspected from 2000 to 2002. Some slight corrosion was found. But for the hangers of the Ohnaruto Bridge, serious corrosion was found on the girder side fixing section as shown in photo 2.8. The structure of the hanger and the cross section of the C.F.R.C. rope are shown in Fig.2.11. This corrosion was very difficult to detect.

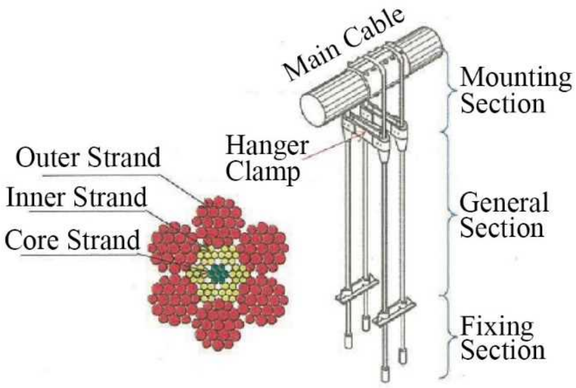


Fig.2.11 Hanger and C.F.R.C rope cross section[14)]

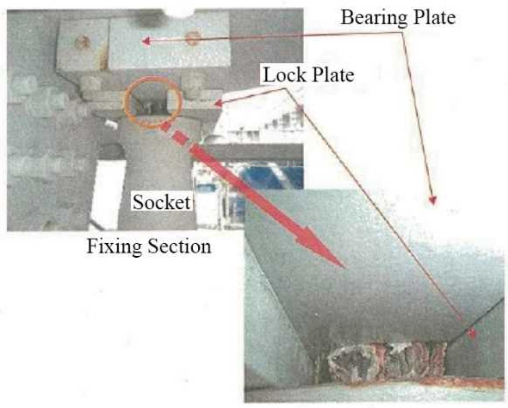


Photo 2.8 Rust of Hanger Fixing Section[14)]

As a policy of the Honshu-Shikoku Bridge Expressway Company, if the sectional decrease of hangers is more than 20%, the hangers shall be replaced. In 2013, three hangers with more than 20% decrease were found and in 2014, these three hangers were replaced. Afterwards, three removed hangers were closely examined to confirm the residual strengths. As a result of tensile test, the tensile strength of hangers with 20% sectional decrease is about the half of the original strength, which coincides with the preceding test results. As a result of site investigation, the corrosion of hangers occur always only near the sockets.

As this problem will continue, a careful observation of corrosions of hangers is indispensable. If badly corroded hangers are found, they need to be replaced by new hangers.

2.4 CABLES OF CABLE-STAYED BRIDGES

2.4.1 DAMAGES TO CABLES OF CABLE-STAYED BRIDGES[16)]

For the cables of the cable-stayed bridges of the Seto Ohashi Bridge, i.e. the Hitsuishijima Bridge and the Iwakurojima Bridge, two rows of vibration suppressor ropes are installed. (Photo2.9) These suppressor ropes restrict the movement of the cables and heighten the natural frequencies of the cables to let the cables vibrate not by frequency of the main span, but by the frequency of the so-called sub-span vibration. Also, the hysteresis effect, the effect to absorb vibration energy of the suppressor ropes is expected.

Photo2.9 Hitsuishijima Bridge and vibration suppressor ropes[1)]

In May 1994, it was found on the Iwakurojima Bridge, one of these suppressor ropes was broken. The broken rope was exchanged quickly and afterwards all suppressor ropes were examined and ropes with larger damages were also exchanged. More damage was found on the Hitsuishijima Bridge in June 1995. The damage of the cable bending buffer, which is placed just above the upper chord of the bridge deck, was found. The detail of the buffer is shown in Fig.2.12, and the damage is shown in Photo 2.10. The damage is as follows; The epoxy resin filler, which was injected at the site, was broken. The PE tube of the cable was cracked and the polymeric grouting material, i.e. the polybutadienic polyurethane, with which the inside of the PE tube was filled, had deteriorated and spilled out. Nineteen more buffers of the longest cables were examined afterwards. It was confirmed that cables between the longest cables to the seventh longest cables suffered some damages. It is considered that this damage was caused by the wake galloping vibration of the cables, because two cables are placed in one place for these cable-stayed bridges. This cause of damage to the buffers, the method of repair works and the counter measures to suppress vibrations were investigated.

Several series of cable bending tests with cable bending buffers were performed, but the same damage could not be reproduced. The cause of the damage could not be clarified but the repair method, temporarily adopted immediately after the detection of the damage, worked well. Instead of the hard epoxy resin filler, a softer polybutadiene resin filler was adopted to repair the several buffers[17)].

The damages of the vibration suppressor ropes may continue further because the vibration of cables continues. If larger damages are found, these ropes need to be replaced. The damages of PE tubes at cable bending buffers are almost solved by the countermeasures at the site.

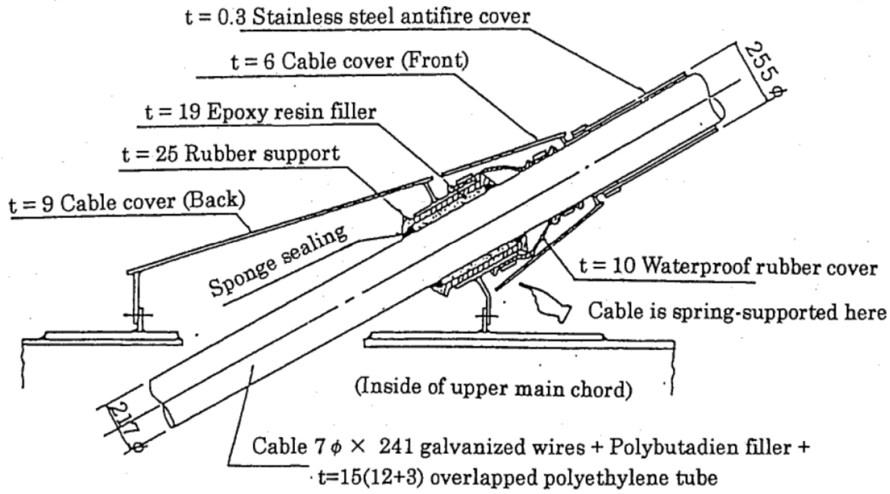


Fig.2.12 Cable bending buffer[16)]

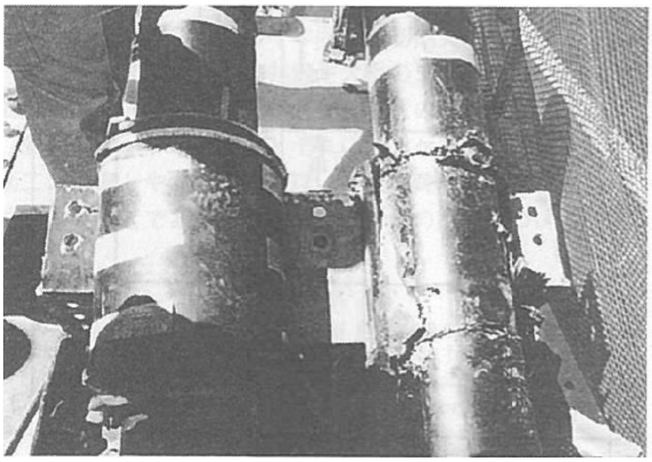


Photo2.10 Damage of cable bending buffers[16)]

2.4.2 VIBRATIONS OF CABLES OF CABLE-STAYED BRIDGES[18)19)]

For the cables of the Hitsuishijima Bridge and the Iwakurojima Bridge, two rows of cables are placed in one place in a parallel position and the wake galloping was confirmed during the construction stage. After the wind tunnel test and the structural investigation study, it was decided to install vibration suppressor ropes and spacers as shown in Fig.2.13, to suppress the vibrations of 1st to 7th modes which were expected to occur in the wind speed range of about 5m/s to 30m/s.

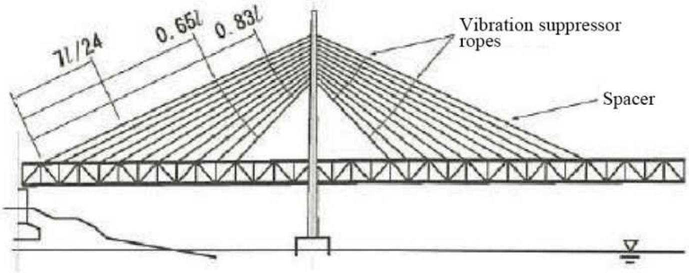


Fig.2.13 Countermeasures against wake galloping[18)]

In 1994, a breakage of vibration suppressor rope was found after a typhoon hit the area. During the period of about 10 years after this incident, a few more breakages of wires were observed. The cause of these breakages were thought to be the sub-span wake galloping of the cables whose nodes are at the positions of the vibration suppressor ropes and the spacers, as shown in Fig.2.14. As the amplitude of the vibrations were within the anticipated range from the beginning so that no counter measures were taken except for the observations.

After the application of the helical wires on the hangers of the Akashi Kaikyo Bridge, the applicability of the helical wires on these cables was examined by the two dimensional rigid model wind tunnel tests. It was found that 12mm diameter helical wire with 650mm winding pitch over the 187mm diameter cables was the most effective. (Fig.2.15) The 187mm diameter cables are the longest cables of the Hitsuishijima Bridge and the Iwakurojima Bridge. But at present, this countermeasure is not applied to the actual bridges. The situation of the cable vibrations is being carefully observed.

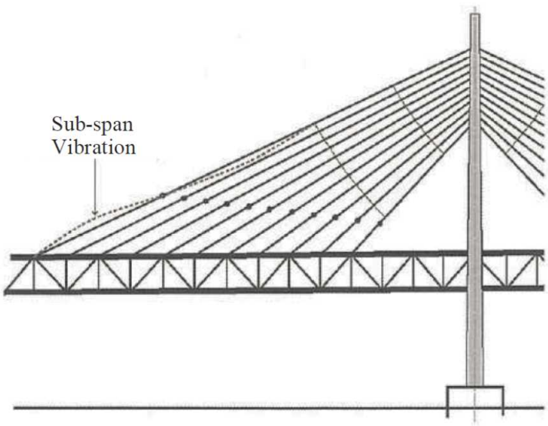


Fig.2.14 Sub-span Vibration of Cable[19)]

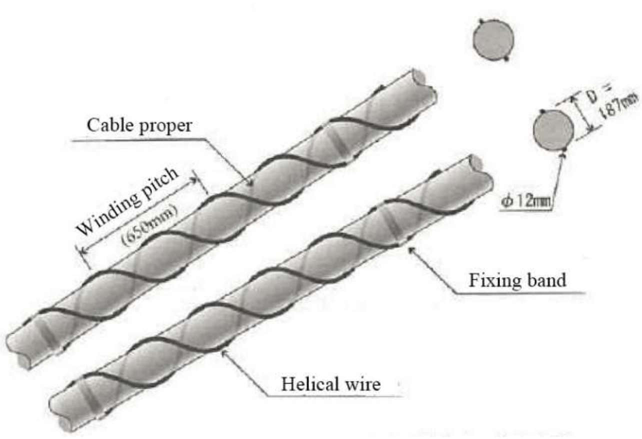


Fig.2.15 Helical wires for parallel cables[18)]

2.4.3 DAMAGE AND REPAIR OF CABLES OF BINH BRIDGE IN VIETNAM[20)21)]

On July 17th 2010, the Binh Bridge, a cable-stayed bridge in Haiphong, was hit by a ship, which was washed away by the typhoon wind from the nearby port where the ship was moored for a repair. The Binh Bridge was completed in 2005 as a Japanese ODA project. The bridge is shown in Photo 2.11. The accident is shown in Photo 2.12. The upper part of the ship collided against one of the edge girders of the bridge deck and damaged the edge girder and the cables.

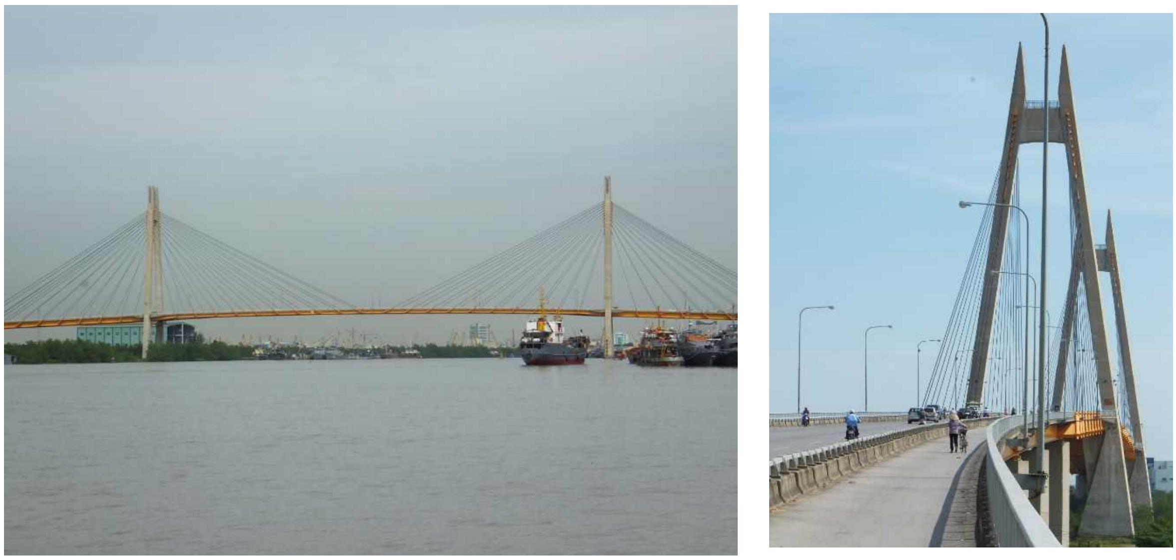


Photo 2.11 The Binh Bridge in Haiphong, Vietnam

Photo 2.12. Ship collision against edge girder[20)]

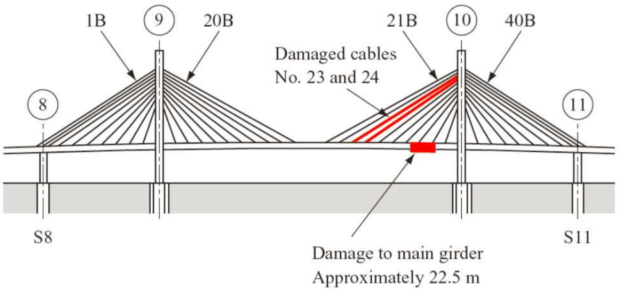


Fig. 2.16 Damages of the Binh Bridge[21)]

The damages of this accident are illustrated in Fig. 2.16 and the damages of cables are shown in Photo 2.13 and the damages of girders, in Photo 2.14.

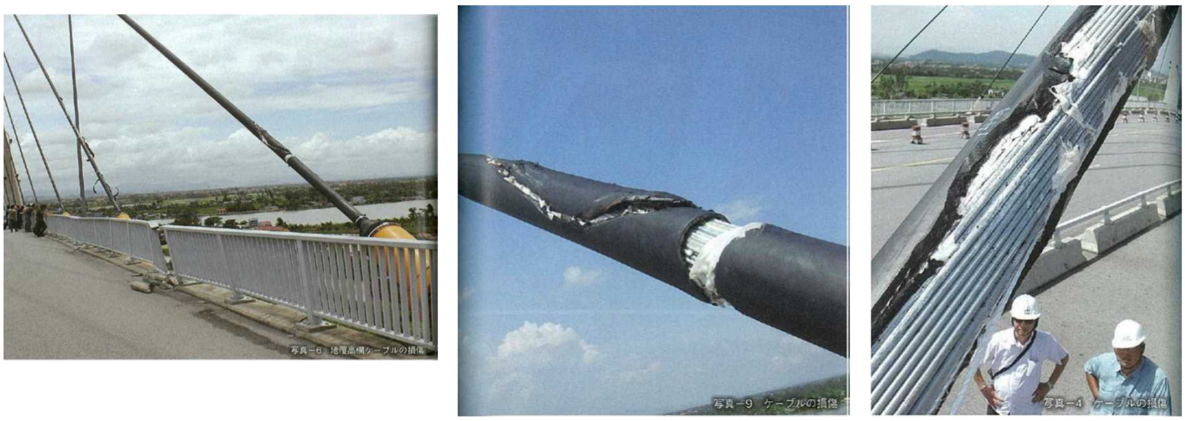


Photo 2.13 Cable damages[20)]

Photo 2.14 Girder damages[20)]

The damaged parts of the girder was cut out and replaced by the new parts. The new parts were welded to the original remaining girder. To achieve this replacement, first the temporary triangular shaped bypass truss girder was fixed under the edge girder and the compression force borne by the damaged lower flange was transferred to the temporary bypass truss girder. Afterwards the damaged parts along the lower flanges of the edge girder were removed and replaced by the newly fabricated same shaped lower flange parts. The new parts were welded to the remaining original girder.

The two damaged cables were replaced as a result of the investigation and consideration. The accident occurred on July 17th 2010 and the repair work began in May 2012 and ended in December 2012. On the two damaged cables, no wire breakage was confirmed and there should have been no reduction of the strength. The scratches were only on the surface layer wires and the inside wires were without scratches. However, the surface scratches and the surface corrosion might influence the fatigue strength. The durability of the cables directly affects the durability of the whole cable-stayed bridge so that it was suggested to the Vietnamese side that the damage of the cables needed to be treated immediately and a proper counter measures were needed. The white rust were confirmed on the wire surfaces which were exposed to the outside due to the breakage of PE tube covers and the rain water. It was feared some rain water stayed at the bottom of the cable. The zinc layer (more than 300g/m^2) of the galvanized wire could disappear easily if it was left untreated. If the zinc layer would be lost, the wire would be corroded consequently and the fatigue strength could be lowered. The correct evaluation of the cable durability was difficult to predict and the long term cable durability could not be assured so that the cable replacement was suggested and the Vietnamese side accepted the suggestion. Then two damaged cables were replaced by two new cables. The cable replacement due to the accident might be a rare case in the world.

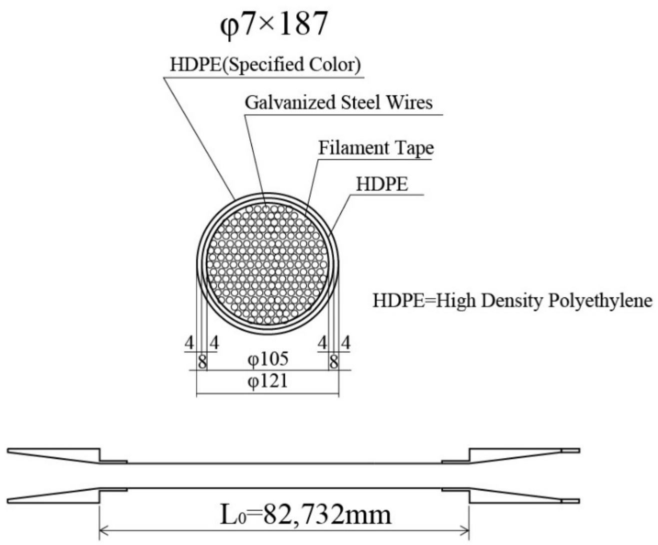


Fig.2.17 Cable length L_0=82,732mm

2.4.4 ZINC CORROSION OF STAY CABLES

The following example is from one cable-stayed bridge in Vietnam. The corrosion resistance of PWS cable for cable-stayed bridges was examined when one cable surface was damaged by a hook of crane during the construction. The inside wires were not damaged but only the outer PE cover was damaged and a small part of cable wires was exposed.

The dimensions of damaged cable are shown in Fig.2.17. The inside diameter is φ105mm. Inside of this cable there are 187wires with diameter of φ7mm. There are many voids among them. The total cross sectional area of voids can be calculated as follows;

$$\mathrm{S} = \frac{\pi \times 105^2}{4} - 187 \times \frac{\pi \times 7^2}{4} = 1{,}462mm^2 \tag{2.1}$$

The cable length between sockets is L_0=82,732mm

Then the total volume of voids is,

$$\mathrm{V} = \mathrm{S} \cdot L_0 = 1{,}462mm^2 \times 82.732mm = 120{,}988{,}218mm^3 = 120{,}988cm^3 \tag{2.2}$$

The void is filled with air. If it is assumed that the cable is fabricated at the temperature of 10℃, at 0℃, the volume shrinks accordingly. The volume at 0℃ can be calculated as follows;

$$\mathrm{V}(0℃) = 120{,}988 \times \left(1 - \frac{10}{273}\right) = 116{,}556cm^3 \tag{2.3}$$

(This 10℃ assumption is in the safer side. At 10℃, larger amount of oxygen is contained. At 30℃, less amount of oxygen is contained inside of the PE tube.)

1/5 of the air consists of oxygen. Therefore the volume of oxygen is

$$V_{O2} = \frac{116{,}556}{5} = 23{,}311cm^3 \tag{2.4}$$

At 0℃, 1 mol of gas material has the volume of 22.4 liter. That is 22,400 cm^3

Then the amount of oxygen inside of PE tube is,

$$\text{Amount of oxygen} = \frac{23{,}311cm^3}{22{,}400cm^3} = 1.041\ mol \tag{2.5}$$

The cable wires are galvanized or zinc coated. The amount of zinc is prescribed as 300g/m^2. The steel wires are galvanized to prevent the corrosion of steel wires. If the galvanized wires are exposed to the severe corrosive circumstances, the zinc will be corroded first because of the order of the ionization tendency. Thus zinc protects the steel wires. The process of oxidization of zinc can be expressed as follows.

$$2\mathrm{Zn} + O_2 \rightarrow 2\mathrm{ZnO} \tag{2.6}$$

2 mol of zinc will react with 1 mol of oxygen. As calculated above, there is 1.041 mol of oxygen inside of the PE tube. Then if 2×1.041=2.082 mol of zinc is corroded, there will be no more oxygen inside of the PE tube and the corrosion process cannot be proceeded anymore. The atomic weight of zinc is 65.38. Then 2.082 mol of zinc is,

$$\text{Zinc weight} = 65.38\mathrm{g} \times 2.082\ \mathrm{mol} = 136.12\mathrm{g} \tag{2.7}$$

If 136.12g of zinc is corroded and turns into ZnO, then the oxygen is completely consumed. Then the question is, 136.12g of zinc corresponds to what amount of surface area of wires. The surface area of one wire can be calculated as follows;

$$\mathrm{S} = \pi \times \mathrm{D} \times 82{,}732\mathrm{mm} = 3.14 \times 7\mathrm{mm} \times 82{,}732\mathrm{mm} = 1{,}819{,}372mm^2 = 1.82m^2 \tag{2.8}$$

As zinc is coated with 300g/m^2, one wire is coated with 300×1.82m^2=546g of zinc. There are 187 wires, the total amount of zinc of one cable is;

$$\text{Totalamount of zinc} = 546 \times 187 = 102{,}102\mathrm{g} \tag{2.9}$$

Compared to 102,102g of zinc, the loss of 136.12g is quite small and still a large amount of zinc will remain after the consumption of the oxygen inside of the PE tube. It can be concluded from this calculation that if the damaged cable surface is well repaired and the air tightness of the cable is assured, there will be almost no possibility of corrosion of steel wires afterwards.

In Sec.2.4.1, the damage of PE tube of the PWS cable is explained. When these cracks were found, the inside wires were quite healthy probably because these wires were not exposed to the outside air for a long time. The cracks of the PE tube were repaired for these cables. Then the airtightness must have been recovered and the corrosion of cable wires must have been prevented. The experiment of repair of PE tube is shown in Photo 2.15. It is quite easy to repair damaged PE tubes in this way.

Photo 2.15 Repair experiment of PE tube

In Sec.2.4.3, the repair of damaged cables of the Binh Bridge is introduced. In this case, it was decided to replace the cables with the new cables on condition that the durability over time of the damaged cables was unclear. The cables were damaged in July 2010 and they were repaired in December 2012, about 2 years after the cables were damaged. After the accident, some white rust was confirmed on the surface of the wires and it was feared some rain water remained at the bottom of cables. But at the time of repair, the surface zinc layer of the wires was not lost. Hence if the broken PE tubes had been repaired properly and the air tightness of the cables had been recovered completely, the replacement of the cables might not have been necessary if the above experimental calculation is correct.

When cable surfaces are damaged and the inside cable wires are not damaged, a repair method needs to be decided. If a damage is smaller which is close to the situation of Photo 2.15 and soon after the damage, this kind of damage should be repaired at the site. The repaired damage will assure the same air tightness as before.

If a damage is large as shown in Photo 2.13, the selection becomes difficult. Even if the damage of PE tube is repaired but still if the recovery of airtightness is difficult, it is necessary to replace whole cables. If it is judged that the air tightness can be recovered, the cables should be repaired at the site. For this decision, a standard for the repair needs to be established. To establish a standard, the airtightness of the original cables, the rate of loss of surface zinc along the time, the air tightness of the repaired cables, etc. needs to be confirmed, if necessary, by experiments.

2.4.5 CABLE FIXING STRUCTURE OF BRIDGE DECK OF TATARA BRIDGE[22)]

The pipe cable fixing structure was adopted for the cable stayed bridges of the Meiko-Nishi Bridge of the Japan Highway Public Corporation and the Ikuchi Bridge of the Honshu-Shikoku Bridge Authority. The cable fixing structure of the Ikuchi Bridge is shown in the photos (Photo 2.16 and 2.17).

Photo 2.16 Pipe Fixing Structure below Girder[1)]

Photo 2.17 Pipe Fixing Structure inside of Girder[1)]

The concept of the pipe fixing structure is simple because the pipes are welded between the webs of the main girder as shown in the Fig.2.18. The tension from the cable is first accepted by the pipe and the lifting force is transmitted from the pipe to the web directly as they are welded together. Then the web plates support the main girder

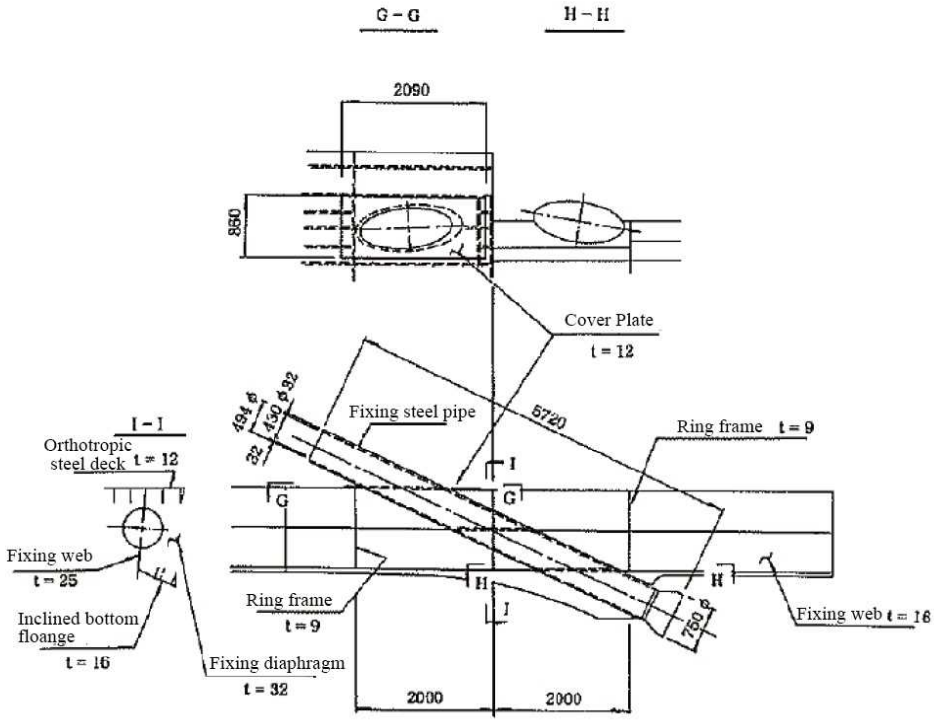


Fig.2.18 Cable Fixing Structure of the Ikuchi Bridge[23)]

The center span of the Tatara Bridge is 890m and this was the world's longest at the time of completion in May 1999. When the Tatara Bridge was planned, new cable fixing structures were investigated to improve the box girder appearance as the pipe fixing structure protrudes from the girder and the aesthetic appearance might not be so high.

Two types of cable fixing structures were proposed and they were compared through the fatigue strength test. Two types of fixing structure are shown below.

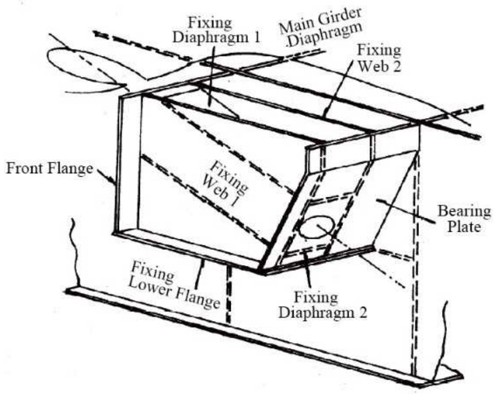


Fig.2.19 Fixing Beam Alternative[22)]

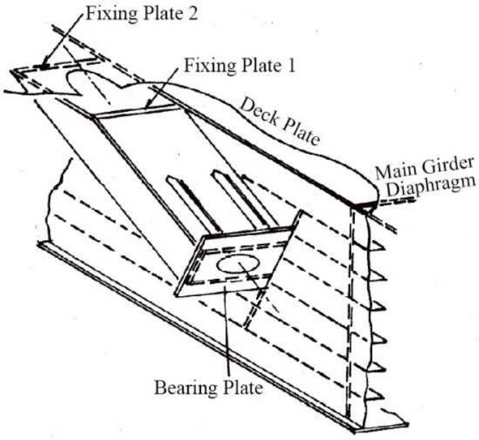


Fig.2.20 Fixing Column Alternative[22)]

The structure of the fixing beam alternative is as follows. First the cable tension is accepted by a small box and then the box is fixed to the girder as a short cantilever beam between two diaphragms of the main girder. The variation of the cable angles can be adjusted by the variation of the bearing plate angles.

The structure of the fixing column alternative is as follows. The pipes of the pipe fixing structure are embedded in the webs of the main girders as shown in Photo 2.17 and Fig.2.18. Instead of pipes, boxes are attached to the side of webs. The end bearing plates of boxes accept cable tensions and the boxes transmit the cable tensions to the web of girders by shear forces. This structure is simpler than the fixing beam alternative.

These two types were compared precisely for their effectiveness of force transmission to the main girders, the easiness of fabrication and the fatigue strengths.

As a result of comparison, both types were feasible as cable fixing structure of the Tatara Bridge. Some reinforcements were recommended to increase the fatigue strengths of both fixing structures. Finally, because of the simplicity of the flow of the force from the fixing structure to the girder and the easiness of fabrication, the fixing column alternative was selected and adopted for the Tatara Bridge. At the detailed design stage of the Tatara Bridge, FEM analysis was performed for the closed rectangular column shown in Fig.2.20[24)]. As a result it became clear that the outside fixing rib shown in Fig.2.21 does not share a large amount of stress. To facilitate the fabrication and the maintenance work, the outside fixing rib was cut-open to have an oval-shaped hole. The actual cable fixing structure of the Tatara Bridge is shown in Photo 2.18.

If this structure and the pipe fixing structure of the Ikuchi Bridge are compared, this structure may be better from the view point of the maintenance. The inside of the fixing column of this structure can be inspected but the inside of the pipe structure cannot be inspected. If water is accumulated inside of the pipe, it is difficult to detect.

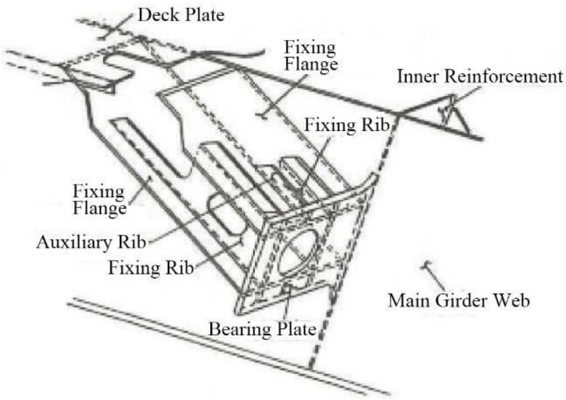


Fig.2.21 Fixing Column after Detail Design[24)]

Photo 2.18 Cable fixing structure of Tatara Bridge[1)]

2.5 EXPANSION JOINTS[16)]

2.5.1 LINK EXPANSION JOINT

The truss bridges and the cable-stayed bridges of the Seto Ohashi Bridge employ the link expansion joints. The Hitsuishijima Bridge, the Iwakurojima Bridge, the Yoshima Bridge and the Bannosu Truss Bridge, the link expansion joints are installed. For these joints, the noise problem became evident after the opening of the Seto Ohashi Bridge in April 1988. Then three joints, two on Iwakurojima Island and one on Yoshima Island, were improved to reduce the noise, but this could not be a fundamental solution. From 1993, the structure of the joint was changed totally. The discrete fingers of the joint were the noise sources, for the new joint, these fingers are cast as one structure. The pin connection of the fingers has been changed to the rubber connection. Until 1997, almost all of the link expansion joints were exchanged to the new link expansion joints. The improved link expansion joint is shown in Fig.2.22, the improvements are summarized in Fig.2.23. In Fig.2.24, the mechanism of noise source is illustrated. In Fig.2.25, the comparison of the noise of the former joints and the new joints is shown, The noise of the joint of the 3P pier of the Bannosu Truss Bridge was measured on the ground level. From Fig.2.25, the effect of the improvement of the joint is evident.

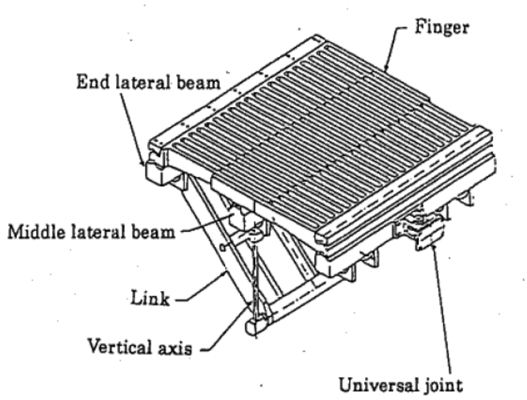


Fig.2.22 Improved link expansion joint

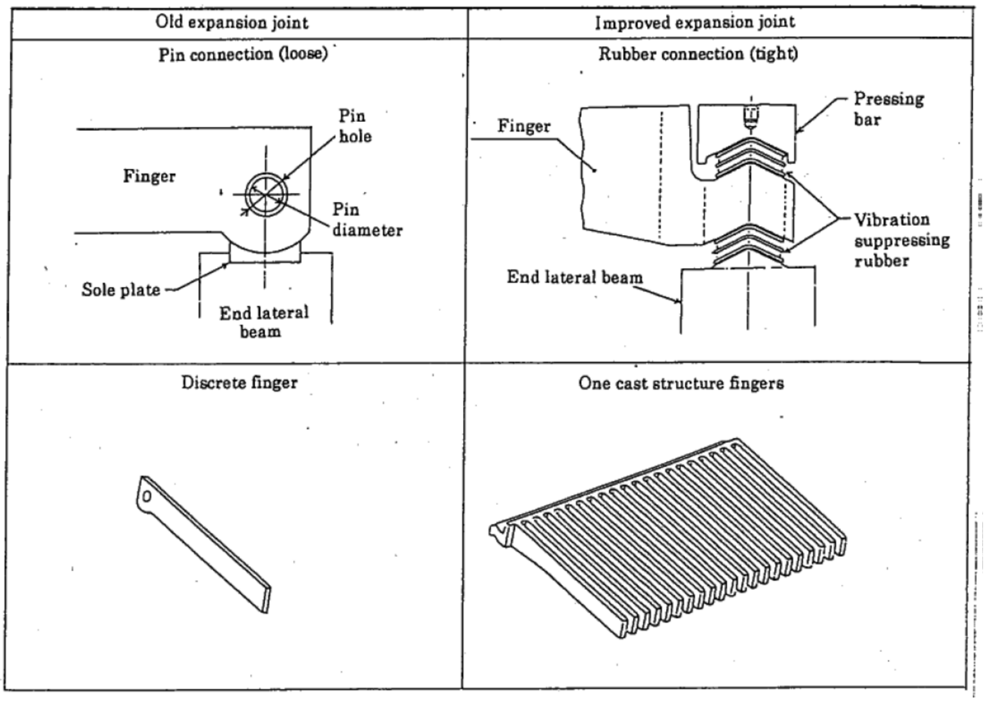
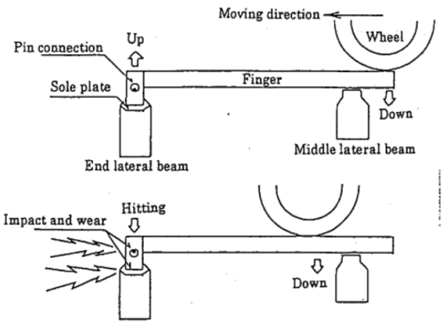


Fig.2.23 Improvement of link expansion joint



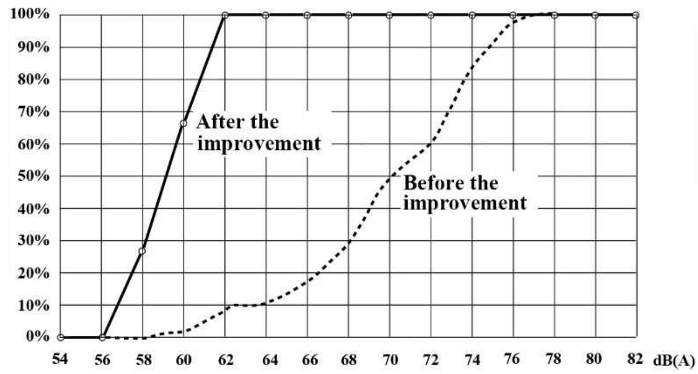


Fig.2.24 Mechanism of noise source

Fig.2.25 Comparison of Cumulative Distribution of Noise

Another improvement to this joint is that the prevention device against the inclination of this joint is added. In May 3, 1994, there was a failure that one unit of the link expansion joint placed on 4P pier of the Iwakurojima Bridge and on the driving lane of the up lane, was inclined and the lane became impossible to pass. The failure of this joint is shown in Photo 2.19. The cause of the failure is that both nuts of bolts which connect the expansion joint and the universal joint of the bridge deck became loose and detached. Consequently one end of the expansion joint fell and the whole joint was inclined. In Fig.2.26, the relations of bolts, nuts and a universal joint is shown. To prevent this kind of accident, the improvements of the expansion joint shown in Fig.2.27 are proposed. These improvements are as follows; the nuts and bolts are marked together so that the loosening of nuts can easily be detected, a cotter pin is applied on each bolt so that even if a nut is loosened, a nut will not fall out finally, stoppers are added on vertical supports of expansion joints. With this stopper, even if nuts are loosened and detach the expansion joint will continue to be the same position, and the accident will not occur. All of the link expansion joints were improved in this manner until April 1995. Because of these improvement, the problems of the link expansion joints are solved almost completely.

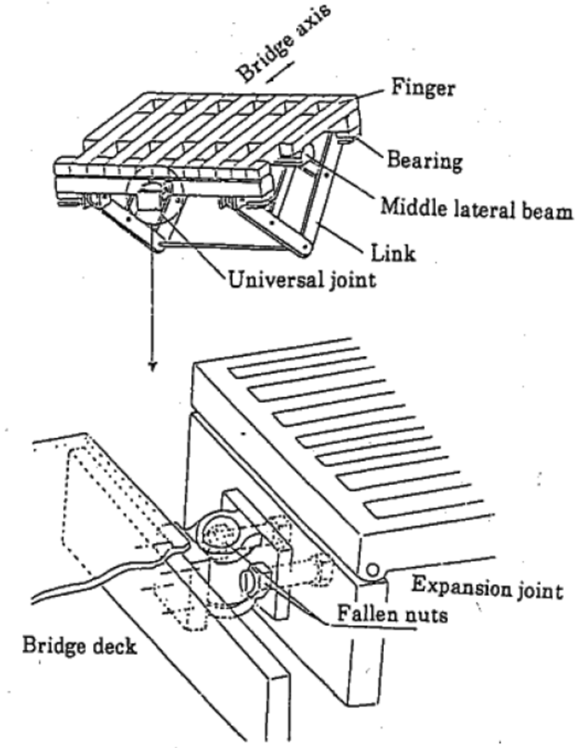


Fig.2.26 Fallen nuts and universal joint

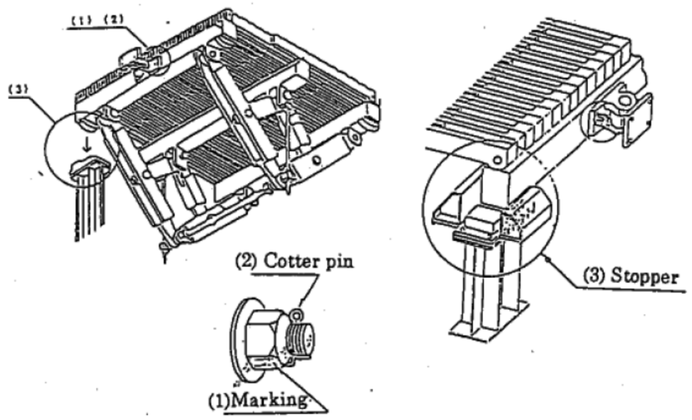


Fig.2.27 Prevention devices against inclination of joint

Photo 2.19 Accident of link expansion joint

2.5.2 ROLLING LEAF EXPANSION JOINT

Three suspension bridges of the Seto Ohashi Bridge employ the rolling leaf expansion joints. For the Shimotsui-Seto Bridge, the Kita Bisan-Seto Bridge and the Minami Bisan-Seto Bridge, the rolling leaf expansion joints are installed. The structure of the rolling leaf expansion joints is shown in Fig.2.28. For these joints, too, the noise problem arose among the residents after the opening of the Seto Ohashi Bridge.

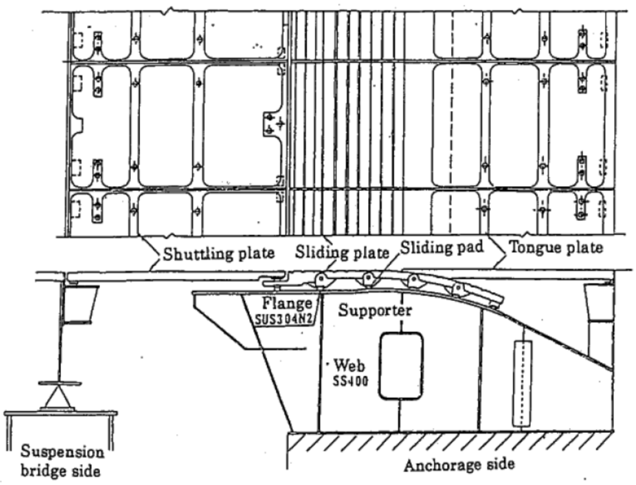


Fig.2.28 Structure of rolling leaf expansion joint

To solve the problem, the expansion joint on SBA3 pier of the Shimotsui-Seto Bridge on Honshu Island and the expansion joint on 1A anchorage of the Kita Bisan-Seto Bridge on Yoshima Island were improved from April 1988 to March 1989. The expansion joint of the Shimotsui-Seto Bridge was first improved. The Major improvements were as follows. Mortar was poured into the box shaped supporters. A whole expansion joint was closed completely by the noise insulator and the glass wool as a sound absorbing material, was placed inside. For the Kita Bisan-Seto Bridge, as the latter improvement was more effective, only the latter improvement was adopted.

After these improvements, the expansion Joint of the Shimotsui-Seto Bridge was further improved by smoothing the surface of the joint. After these works, there were no problems with this expansion joint.

In March 1995, cracks were found on the stainless steel flange of the supporter of the expansion joint on SBA3 pier of the Shimotsui-Seto Bridge. As sliding pads of sliding plates move on the flange of the supporter, the flange cannot be painted. This is why stainless steel is adopted for the flanges. After the rolling leaf expansion joints were fabricated by welding at the shop and the shapes were corrected by heating, a whole unit of each expansion joint was annealed to remove residual stresses. Except for the flanges, all other parts of the supporters are made of SS steel i.e. rolled steel for general structures, the coefficient of linear expansion of SS steel and that of stainless steel are different by more than 30 percent (Stainless steel: 14.7×10^{-6}, SS steel : 10.7×10^{-6}). Because of this difference, tensile stresses due to the thermal load remained inside of the stainless steel. Stress corrosion cracks can occur easily when the steel plate is under the tensile stress. The repair of cracks by welding was first considered, but this method took a long time at the site, consequently the time of traffic restriction would continue long. Therefore this method was not adopted. Instead, the removal of the corrosive environment and the alleviation of stress level of the flanges were adopted. To shut out the corrosive environment from flanges, 2mm thick stainless plate was pasted on the surface of a flange by spot welding. The edges of flanges were painted. To reduce the stress level of the flange, Shrinkage-compensating mortar was poured into the inside of the box shaped supporters until mortar reaches the bottom surface of the flange. By this improvement, the bending stress would not occur in the flanges. For the parts of flanges which extruded out of the webs of the supporters, reinforcing attachments were welded on the webs of the supporters. Shrinkage-compensating mortar was then poured just in the same way as the inside, which would help reduce the stresses of the flanges. These repair works are shown in Fig.2.29. In 1996, the expansion joints of SBA4 pier of the Shimotsui-Seto Bridge and those of 4A anchorage of the Kita Bisan-Seto Bridge were repaired. In 1997, SBA3 of the Shimotsui-Seto Bridge and 1A anchorage of the Kita Bisan-Seto Bridge were repaired. The rolling leaf expansion joints were repaired and improved at the site and at present, no problems are reported.

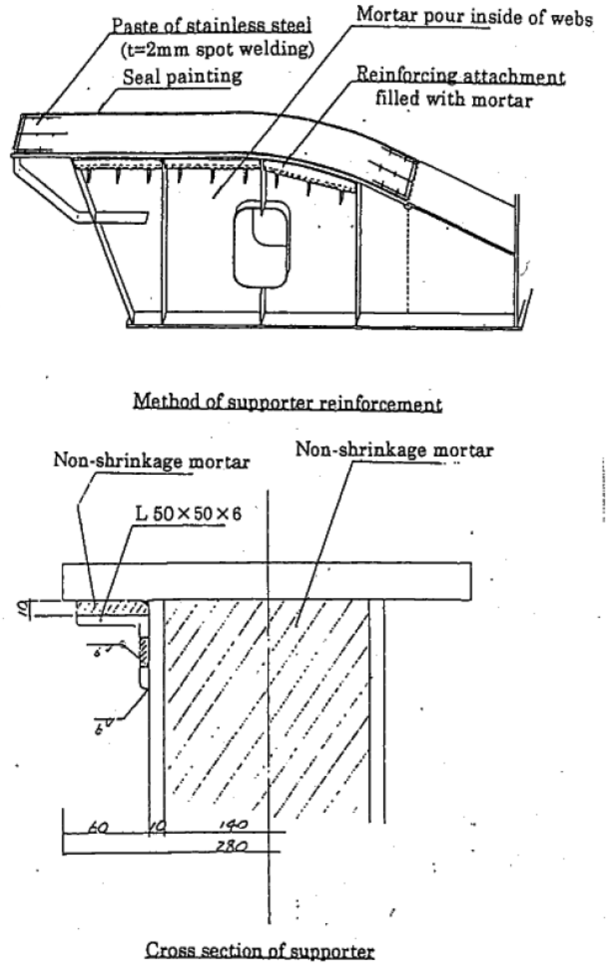


Fig.2.29 Repair works of rolling leaf expansion joint

2.6 NOISE PROBLEM OF END LINKS[16)]

The ends of stiffening truss girders of the three suspension bridges of the Seto Ohashi Bridge are supported upward by end links vertically from beneath the girder. The maximum reaction force of the end link of the Shimotsui-Seto Bridge is 2558 ton, which is about twice as large as other two suspension bridges because this bridge has extended side spans which are not suspended from the main cables. From June 1993, these end links began to make large noises when heavier freight trains passed through the bridge. The general drawing of the Shimotsui Seto Bridge and its end link are shown in Figs.2.30 and 2.31, From January to February 1994, the noise source was closely investigated using the acoustic emission method. It was found that the noise source was the center of the semispherical boss of the lower pivot structure of the end links. The pivot structure consists of a semispherical boss which is embedded on a concrete abutment and a concave semispherical cover which is placed at the end of the link beam and over the boss. It was inferred that the movement of a pivot was not large enough to diffuse solid lubricant embedded in the concave cover, consequently a stick slip phenomenon of metal surfaces occurred and noises were emitted. To prevent this stick slip phenomenon, a hole was bored on a concave cover to reach the center of a boss and anti stick slip oil with high permeability was pressured into the surfaces. This countermeasure worked well. Consequently the noise of the east end link of SBA3 pier was reduced by 50 percent and that of the west Iink, by 10 percent. The reduction of noise of the east end link of SBA3 pier is shown in Fig.2.32. All of the end links of the Shimotsui-Seto Bridge were repaired by this method until 1997.

The end links were improved at the site and the noise problems were solved. If this kind of end links is planned in the future, the diffusion of solid lubricant embedded in the concave cover needs to be carefully designed so that the stick slip phenomenon of metal surfaces will not occur.

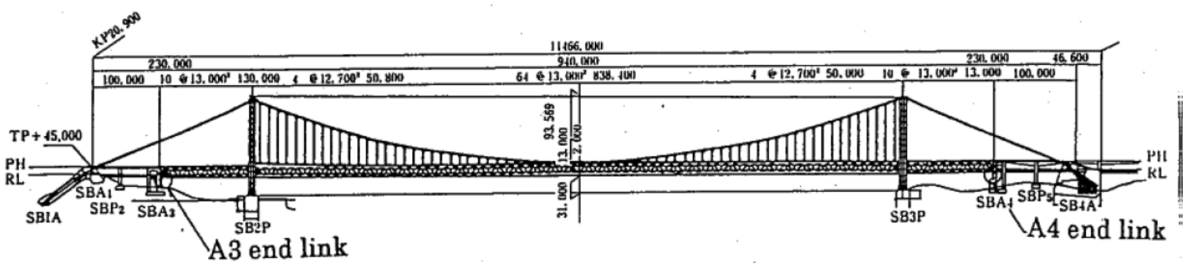


Fig.2.30 General plan of the Shimotsui Seto Bridge

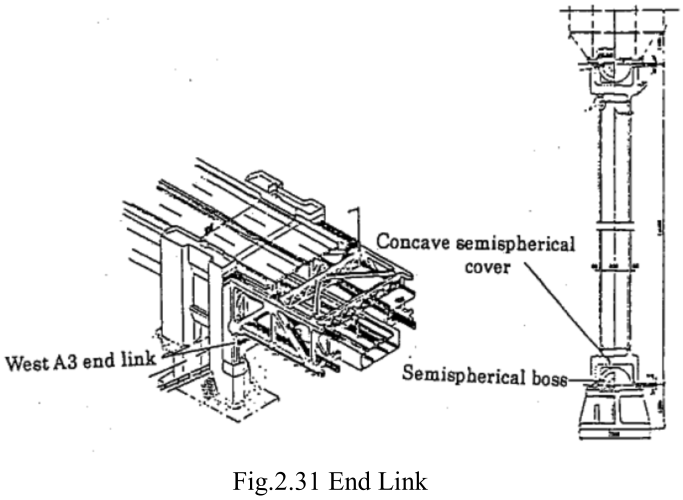


Fig.2.31 End Link

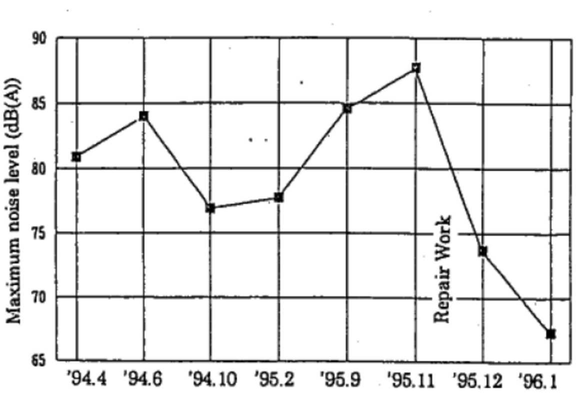


Fig.2.32 Reduction of noise of east end link of SBA3

2.7 ORTHOTROPIC STEEL DECK PAVEMENT[25)]

All of the suspension bridges and the cable-stayed bridges of the Honshu-Shikoku Expressway Company adopt the orthotropic steel decks to reduce the dead weight. Although the orthotropic steel deck is light, it is easily deflected by the axle loads and the water seepage on the deck will rust the steel surfaces. To cope with this condition, the combination of the Gussasphalt for the base course and the modified asphalt for the surface course is extensively adopted. The Gussasphalt can follow the deflection of steel decks easily and has a higher flow resistance and a higher crack resistance. The Gussasphalt has a high waterproof function to eliminate water from the steel deck, too.

For the conventional asphalt pavement, the deteriorated pavement surface is milled and the surface is overlaid. Or the total pavement is removed and the new pavement is constructed. But for the orthotropic steel deck pavement, a high temperature asphalt material which reaches as high as 240℃ needs to be transported and constructed at the site. This requires the careful quality control to consider the deflection of the steel decks and the influence to the paint surface, which results in a higher construction cost. Hence the Honshu-Shikoku Bridge Expressway Company tries to preserve the Gussasphalt as long as possible by preserving the modified asphalt surface course. To preserve the surface course, MS(Micro Surfacing) method is being adopted. 5mm thick deteriorated surface is milled and the surface is paved with 5mm thick slurry. By this method, the quality of the modified asphalt can be preserved longer and consequently the Gussasphalt base course can be preserved longer. The Gussasphalt method is extensively adopted for the bridges of the Honshu-Shikoku Bridge Expressway Company and there are no major problems. This method is well established in Japan.

2.8 INSPECTION VEHICLES FOR LONG SPAN BRIDGES[26)]

For the suspension bridges with stiffening truss girders of the Honshu-Shikoku Bridge Expressway Company, the outside girder inspection vehicles, the inside girder inspection vehicles, the tower inspection vehicles and the cable inspection vehicles are equipped.

The first example of the inspection vehicle for the stiffening truss girder in Japan was the under girder inspection vehicle for the Kanmon Bridge. This under girder inspection vehicle was flat and from the vehicle, only the lower truss chords and the lower lateral truss chords could be accessed. (In recent years, the vehicle of the Kanmon Bridge was improved to assure a higher accessibility to other members.) As the second example of the inspection vehicle, for the Innoshima Bridge and the Ohnaruto Bridge, the U-shaped under girder inspection vehicles were installed. From the U-shaped vehicles, it is possible to access the side faces of the truss girders.

The U-shaped under girder inspection vehicle was further improved and telescoping steps from which the inside of the girder can be inspected, were added. This step could move upward and downward along the vertical shaft of the U-shaped vehicle. The improved U-shaped under girder inspection vehicles are introduced to the suspension bridges of the Seto Ohashi Bridge and the Akashi Kaikyo Bridge. (Photo2.20)

The floor width of the under girder inspection vehicle of the Ohmishima Bridge (Steel Arch Bridge) completed in 1979 was only about 2m but this width became 6m from the under girder inspection vehicles of the suspension bridges of the Seto Ohashi Bridge and 6m width became the standard width.

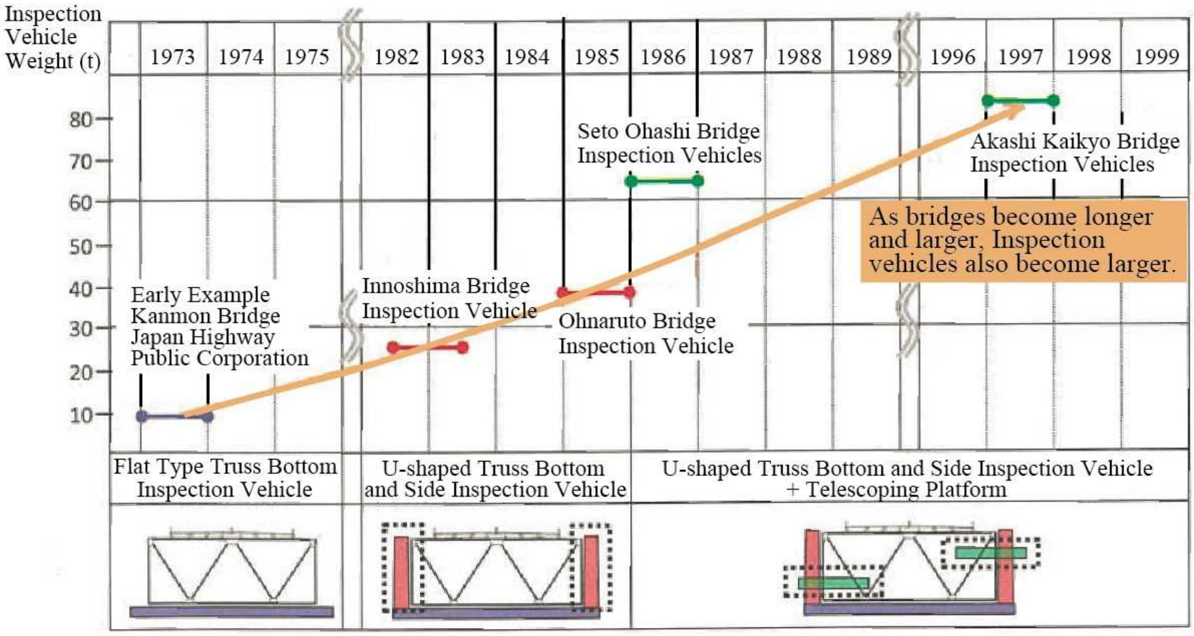
Photo 2.20 U-shaped under girder inspection vehicle of the Minami Bisan-Seto Bridge[25)]

As the bridge became larger and to improve the accessibility to the truss members to facilitate the maintenance work, the size of the inspection vehicle also became much larger. This tendency is shown in Fig.2.33.

These maintenance vehicles are electrified and the trolley lines are installed along the whole length of the girders or the cabtire cables are utilized. As the trolley lines or the cabtire cables need to be prepared for the whole length of the girders, the initial cost is higher. The trolley lines are easily deteriorated by the flying salinity and the maintenance cost is also high. Therefore for the recent bridges, i.e. the Kurushima Kaikyo Bridges and the Tatara Bridge, lead batteries are utilized to move the inspection vehicles. These batteries are charged when the inspection vehicles stop at the end of girders. For the inspection vehicles of the Tozaki Viaduct, the trolley lines were dismantled and the lighter lithium batteries were introduced because the lead batteries were heavier and exceeded the limit weight of the inspection vehicle.

The inspection vehicles of the Honshu-Shikoku Bridge Expressway Company are very large to achieve the higher accessibility to the truss members but this size of inspection vehicle may be difficult to adopt for the inspection vehicles in developing country. The under girder inspection vehicle of the Matadi Bridge is a similar type of vehicle of the Kanmon Bridge. But it is not electrified.

The inspection vehicles in Japan are being improved to achieve the higher accessibility and the better power supply system to move the inspection vehicles.



Seto Ohashi Bridge : Suspension Bridge: Shimotsui-Seto Bridge, Kita and Minami Bisan-Seto Bridges,
Cable-Sayed Bridge: Hitsuishijima Bridge, Iwakurojima Bridge.

Fig. 2.33 Increasing size of under girder inspection vehicle

2.9 PROBLEMS IN DEVELOPING COUNTRIES

Based on the various experiences of maintenance problems discussed in this chapter, the maintenance problems of long span bridges in developing countries, which will be investigated further in Chapter 3, 4, 5, are selected in this section. The present situation of these maintenance problems discussed in this chapter are summarized in Table 2.2. In the table, 11 problems are listed. Some problems are already solved from the engineering point of view but some are still under the investigation.

Long span bridges consist of many parts. Each part is closely inspected and monitored regularly in Japan. If some defects are found at the site, the phenomenon is closely investigated and analyzed to find out the countermeasures. The defects will be repaired with the suitable method at the site based on the analysis. Then the same analysis will be utilized to improve the performance of the part. The improved new parts will be utilized for the new long span bridges. For example, when the link expansion joints were utilized, the falling-off problem and the noise problem happened. The countermeasure against the falling-off problem was executed at the site. The noise problem was analyzed and the new improved noiseless link expansion joint was developed and installed. This improved link expansion joint is later utilized for the new bridges.

This defect-countermeasure-improvement cycle works well in Japan or in developed countries. But some defects are not completely solved so that a continuous monitoring and a maintenance work are indispensable.

In developing countries, it is very difficult to achieve the above cycle. The major problems may need to be solved by the aid of developed countries. To facilitate the maintenance works, the method of easier maintenance works and the durable facilities are needed more than in developed countries.

Table 2.2 Present situation of maintenance problems

No		Item	Present understanding
1	Main cable	Main cable corrosion protection	For the new bridges, the introduction of the dry air injection system for the main cables and the anchorages, utilizing S-shaped wrapping wires is established. For the repair of existing bridges, the dry air injection system is introduced for cables and anchorages.
2	Hangers	PWS Hangers (Akashi Kaikyo Bridge)	The vibration problem occurred because of the round shaped PWS hangers and the wake flatter. Helical wires were invented and installed. The vibration was suppressed.
3		C.F.R.C. Hangers (Ohnaruto Bridge)	C.F.R.C rope was adopted for the hangers. The corrosion problem happened and the condition is monitored at present. Some hangers were already replaced. For C.F.R.C. rope, vibration problem did not occur.
4	Stay cables	Cables of Cable-stayed Bridges	Vibration problem occurred due to wake galloping and the vibration suppressor ropes were broken and replaced. The counter measure utilizing helical wires was invented but it is not adopted yet. At present, the vibration is kept to be monitored.
5		Damage and repair of cables (Binh Bridge in Vietnam) and Zinc corrosion of cable wires	Cables are damaged by the vibration due to wind or car collision. The damage of Binh Bridge was due to ship collision. It was confirmed that the corrosion resistance of PWS cables is high. A cable surface repair method may be feasible.
6		Cable fixing structure of bridge deck of Tatara Bridge	There are many types of cable fixing structure. The structure adopted by the Tatara Bridge is recommended not only from the view point of structure but also from the view point of maintenance. There are many solutions which can be rational.
7	Noise problems of expansion joint and end link	Link expansion joint	The noise problem was solved completely by improving the structure. The fixing structure to the bridge deck was also improved.
8		Rolling leaf expansion joint	Noise problems were solved by the improvement works at the site.
9		Noise problem of end links	The pivot structure at the lower end of the end link consists of a semispherical boss embedded on a concrete abutment and a concave semispherical cover. The noise problem happened due to the stick slip phenomenon between the boss and the cover. The solid lubricant was not diffused well. A hole was bored on a concave cover and the anti stick slip oil was pressured into the surface of the boss. This counter measure worked well and the problem is almost solved. For the new bridges, it is necessary to design the link so that the diffusion of solid lubricant is well established.
10	Pavement	Orthotropic steel deck pavement	Gussasphalt method is utilized extensively in Japan. There are almost no problems. But in developing countries, the repavement of orthotropic steel decks is a big problem. The Gussasphalt method needs special machines but these machines are unavailable.
11	Inspection vehicles	Inspection vehicle for long span bridges	The inspection vehicles become larger and larger in Japan to achieve better accessibility to the truss members. Battery powered inspection vehicles are invented. The structure of vehicles is still being improved. But this kind of vehicles may not be applicable in developing countries because they do not match to the engineer's understandings as well as the budget.

Table 2.2 Present situation of maintenance problems (continued from left page.)

Problems in Japan or in developing countries	Necessity of investigation in developing countries
The countermeasures against the cable corrosion is almost solved in Japan. The system is operated and monitored continuously. But in the near future, electrified machines may need to be replaced. The cable corrosion is suspected in developing countries, too. A simpler method of countermeasure is preferable.	High
This problem is solved by the installation of Helical wires. A PE cable cover with protrusions to have a similar shape as helical wires is being invented.	Middle Vibration problems and corrosions need to be inspected continuously. If problems happen, a countermeasure is needed.
The natural condition of this bridge is the severest among Honshu-Shikoku Bridges. No serious problems are reported from other suspension bridges. But the corrosion of C.F.R.C. hangers needs to be monitored continuously. In the future this problem may arise in developing countries, too.	
The cable vibration needs to be monitored. If vibration suppressor ropes are broken, they need to be replaced. If this replacement becomes too costly, then the helical wires may be introduced. The vibration problems of stay cables often happen and the close observation is needed	Middle Vibration problems and PE cover damages need to be inspected continuously. If problems happen, a countermeasure is needed.
The same method of cable replacement of the Binh Bridge can be adopted in the future, too. But for the minor cable surface damages, a cable surface repair method can be applied. To adopt the method, the repair standard needs to be established. The same methods as in Japan can be adopted in developing countries, too if necessary.	
It is easier to inspect the column fixing structure adopted by the Tatara Bridge. At present, there are no serious problems. The cable fixing structure needs to be inspected regularly in developing countries, too.	
The noise problem and the falling-off problem are almost solved.	Middle Noise problems and functional problems often happens on expansion joints and the bearings. These parts need to be inspected carefully. If problems happen, a countermeasure is needed. Noise problem may be less important in developing countries.
The noise problem is almost solved.	
The noise problem is almost solved at the site. For the new bridges, the design of the diffusion of solid lubricant needs to be improved.	
There are no major problems for the Gussasphalt method and other methods are also developed in Japan. The orthotropic steel deck pavement is a large problem in developing countries. Different pavement methods which do not need special machines are needed.	High
The accessibility to the members is still being improved. Electrified inspection vehicles are extensively utilized. To alleviate the maintenance cost, battery powered vehicles are introduced. The design of the inspection vehicles for the longer service life is very important in developing countries.	High

In developing countries, the understanding of the long span bridge structure by the bridge maintenance engineers is sometimes insufficient due to the insufficient bridge maintenance experiences. Other problems are the difficulty to obtain various machine parts for the maintenance machines, the limited availability of various construction machines at the market, the theft problems, political turmoil of the country, etc. Due to these problems, the maintenance of machines becomes sometimes very difficult. Then the maintenance work by these machines for the bridges becomes very difficult, too.

This situation becomes most evident for the case of inspection facilities of long span bridges as they are placed on the bridges and utilized for a long time. The possibility of deterioration of parts, the theft or the vandalism of parts is high. Even if some parts are deteriorated or stolen, it is very difficult for the maintenance engineers of developing countries to obtain spare parts to repair the damaged inspection facilities. In developing countries, durable maintenance facilities which can be maintained easily, are expected most. This problem is selected and studied further in Chapter 3.

Some corrosions were found on main cables of suspension bridges in Japan. As the suspension bridges constructed by the Japanese ODA adopted the same specification for the main cables, the corrosions of main cables of these bridges are suspected. For the engineers of developing countries, the confirmation of these corrosions and the planning of countermeasures are very difficult because they do not have the experiences nor the knowledges to open cables, to inspect inside of the cables and to plan countermeasures. Then the aid from Japan is needed. This problem is selected for the further study. The condition of corrosions of main cables and the countermeasures suitable for developing countries are further studied in Chapter 4. The main cables of suspension bridges are the most important parts which accept the loads from the stiffening girders and transmit the force to the anchorages, as explained in Chapter 1. It is very difficult to replace these main cables by the new cables even if the main cables are deteriorated severely, as explained in Sec.2.2.7 unlike other members such as hangers of suspension bridges, cables of cable-stayed bridges, expansion joints, etc. which are to some extent replaceable. This is why the problem of the main cable corrosions is adopted for the further study.

No serious damages of hangers of suspension bridges, stay cables of cable-stayed bridges, expansion joints, etc. in developing countries are found at present except for the damages of stay cables of the Binh Bridge in Vietnam which is discussed in Sec.2.4.3. Hangers, stay cables, expansion joints, etc. need to be inspected and monitored continuously.

In developing countries, the availability of various construction machines at the market is often very limited. This causes a big problem for the orthotropic steel deck pavement. The Gussasphalt method was extensively utilized by the Honshu-Shikoku Bridge Expressway Company. Until now, there are not major problems. One exception is the pavement of the Ohshima Bridge[27)]. For this pavement, a blistering phenomenon was observed and 3000 square meters (about a half of the total bridge area of 560m span bridge) were removed and repaved by the same method in 2010. When the pavement was removed, the corrosions were not confirmed. This proved the water tightness of the Gussasphalt method.

In around year 2000 in Japan, Stone Mastic Asphalt Pavement (SMA pavement, hereinafter.) was adopted for the orthotropic steel deck pavements. But for these pavements, comparatively earlier damages within one or two years after the construction were found[28]. If this SMA pavement was adopted for the long span bridges in developing countries, the earlier damages were suspected. Then for the repavement of the bridges, the more reliable Gussasphalt method may be needed. In developing countries, the repavement of the orthtropic steel deck becomes almost always a big problem because of the lack of the necessary special machines for the Gussasphalt method. This was the case of the Suez Canal Bridge in Egypt and the Diosdado Macapagal Bridge in the Philippines[29]. Both bridges are the cable stayed bridges constructed by the Japanese ODA Project. Therefore different pavement methods, which can be constructed by the available facilities of the developing countries, are anticipated. This problem is selected and studied in Chapter 5. The bridges which are studied in Chapters 3 to 5 are shown in Table 2.3.

Table 2.3 Bridges and their names

Foreign Countries	 Matadi Bridge in Democratic Republic of Congo, A single span suspension bridge with stiffening truss girder which can accommodate both one train track on the lower deck and the road traffic on the upper deck. At present no track is placed. Completed in 1983. Span arrangement is 101m+520m+101m=722m	 Suez Canal Bridge in Egypt, A cable-stayed bridge and the only one bridge at present to cross the Suez Canal. This bridge links Cairo city and Sinai Peninsula in the east. The spans are 163m(43m+50m+70m)+404m +163m(70m+50m+43m)=730m There are two intermediate piers on each side span. This bridge has very high clearance of 70m. This bridge was completed in September 2001 as a Japanese ODA project.

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Chapter 3 PROBLEMS AND IMPROVEMENTS OF MAINTENANCE FACILITIES OF A SUSPENSION BRIDGE

3.1 INTRODUCTION

The Matadi Bridge is a suspension Bridge constructed as the Japanese ODA project in 1983. Recently the maintenance facilities of this bridge were investigated precisely and the condition of these facilities were confirmed because several requests from the maintenance engineers of this bridge to repair the maintenance facilities had reached Japan side. After about 30 years of service, various damages and malfunctions were found but these facilities were still utilized for the daily inspection works successfully. This was partly because of the unelectrified maintenance vehicles which were durable and whose parts were difficult to be stolen. In this chapter, the problems of maintenance facilities are analyzed and the countermeasures are proposed taking into account the conditions of developing countries.

Photo 3.1 Matadi Bridge

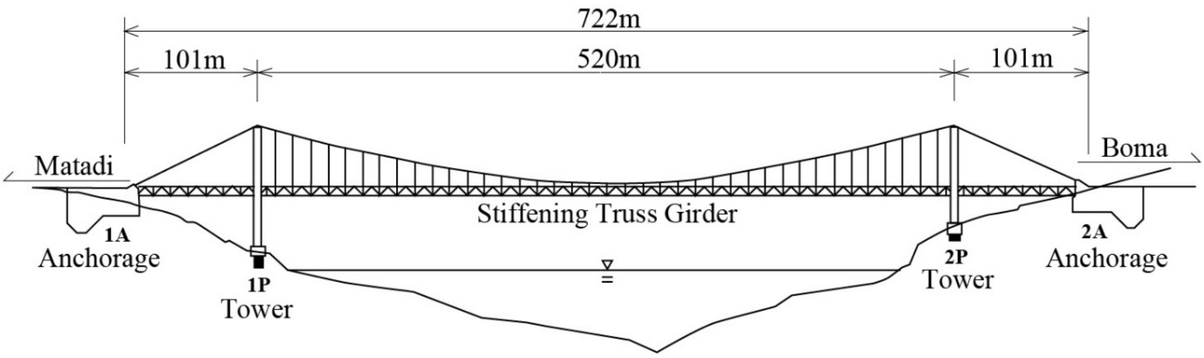


Fig.3.1 General Drawing of Matadi Bridge

The Matadi Bridge, a suspension bridge with the span arrangement of 101+520+101=722 m and a road /railway combined bridge in Democratic Republic of Congo (DRC hereinafter), was constructed by the Japanese ODA and completed in May 1983 (Photo 3.1, Fig.3.1). Since then, the bridge has been maintained by the staff of OEBK (Organization pour L'Equipment de Banana et Kinshasa). The railway was planned to be installed on

Table 3.1 Maintenance history of Matadi Bridge[2)]

Year	Months	Event
2017	May, Apr., Mar., Feb., Jan.	Dry air injection system is being operated.
2016	Dec., Nov., Oct., Sep., Aug.	Dry air injection system was installed for the Matadi Bridge from Nov. 2015 to Mar. 2016. Cable band bolts were also tightened.
2015	Dec., Nov., Oct., Sep.	
2014	Sep., Aug, Jul, Jun, May, Apr, Mar, Feb, Jan	Maintenance facilities were improved at the site. Two under girder inspection vehicles were also added.
		Detailed design of dry air injection system for main cables was executed.
		New Maintenance Manual was established in 2013.
2012	Aug., Jul., Jun., May, Apr.	Cable opening inspection and detailed maintenance facilities inspection were executed in June.
2011	Sep., Aug., Jul., Jun., May, Apr.	Preliminary JICA study for cable inspection and maintenance facilities inspection was performed from June to July.
2010	Aug, Jul, Jun, May, Apr, Mar	Two JICA experts from Honshu-Shikoke Bridge Expressway company visited the bridge from May to June.
2009	Oct, Sep, Aug, Jul, Jun, May, Apr, Mar, Feb, Jan	IHI engineers checked the bridge condition in August.
2003	Dec, Nov, Oct, Sep, Aug, Jul, Jun, May	From 2003 to 2009, OEBK painted the whole bridge by themselves, based on the advice from Japan.
1991	Dec, Nov, Oct, Sep, Aug, Jul, Jun	Four Japanese Experts were dispatched in September for the maintenance activities. But they were forced to return to Japan because of the political turmoil. Afterwards no Japanese experts were dispatched for about 20 years.
		In 1985 a toll system was introduced. 30% of the revenue was allocated for the bridge maintenance.
1983	Dec, Nov, Oct, Sep, Aug, Jul, Jun, May, Apr, Mar, Feb, Jan	Japanese Experts for bridge maintenance were dispatched continually after the completion.
		Maintenance Manual was prepared by the contractor, IHI.
1979	May, Apr, Mar, Feb, Jan	Construction of Matadi Bridge from Feb. 1979 to May 1983.

the lower side of the truss girder and the road was placed on the upper side. The stiffening truss girder is continuous from 1A Anchorage to 2A Anchorage. The side spans are not hung from the main cables. This bridge structure is similar to the Shimotsui-Seto Bridge which has extended spans on the side spans, which are continuous from the center span truss girder and are not hung from the main cables. The Shimotsui-Seto Bridge is also a road /railway combined bridge.

The maintenance history of the Matadi Bridge is shown in Table 3.1[1)]. OEBK has been continuously responsible for the maintenance of the bridge. The arrangement of the continuous and experienced maintenance engineers has also been secured by OEBA. As shown in the table, in 1985, a toll system was introduced to the Matadi Bridge and it was decided to utilize 30% of the revenue for the maintenance of the bridge. Therefore two important factors, the human resources and the fund, were satisfied for this bridge.

After the completion of the Matadi Bridge, Japanese experts were dispatched continually for the maintenance of the bridge. But after 1991, when the Japanese experts were forced to evacuate DRC because of the political turmoil, the Japanese technical cooperation for the bridge maintenance was suspended for nearly 20 years. This shows the necessity of durable maintenance facilities in developing countries.

During this period, OEBK planned to repaint the whole bridge and actually repainted the bridge from 2003 to 2009 by themselves based on the advice from Japanese engineers. From 2009, Japanese technical cooperation was resumed. The bridge condition was checked. Afterwards the maintenance facilities were improved and the dry air injection system for the main cables was introduced as shown in the table.

3.2 MAINTENANCE FACILITIES OF MATADI BRIDGE

The inspection facilities such as the under girder inspection vehicles, the inside girder inspection vehicles, are provided on this bridge. As the bridge has been opened to the public for nearly 30 years, these maintenance vehicles need to be improved because some deteriorations and the consequent problems have been found and reported by OEBK which maintains the bridge.

Based on the maintenance manual provided by the contractor, OEBK routinely inspects the bridge. As the daily inspection and the detailed inspection, they decide the monthly plan and inspect the following parts repeatedly. (a) Stiffening truss girder & steel deck, (b) Expansion joints, (c) Piers, (d) Towers, (e) Cables, (f) Anchorages, (g) Accessories. Out of the 8 items, one item is chosen as the main item to be inspected that month and other items are inspected less intensively. The main focus of the inspection is corrosions. Other important points are, expansion joints which need frequent repair of support beams due to their outward movement, cracks of anchorages, anchorage room cleaning, touch up work, water drain works from the tower, etc. For the longer term periodic maintenance of suspension bridges, there are cable band bolt tension measurements and the bolts retightening, and the surveillance of the bridge shape. These works need special equipment and technology, so that these works are not performed by OEBK. During these inspections and also during the repainting works of the bridge, they fully utilize the equipped inspection vehicles and other facilities, such as the truck crane, the aerial working platform truck, etc. Without them, it must have been almost impossible to do these works. The maintenance manual is recently renewed in cooperation with the Honshu-Shikoku Bridge Expressway Company.

The maintenance equipment of the Matadi Bridge is shown in Fig.3.2. Two under girder inspection vehicles are installed at the center span and they are shown in Fig.3.3, Photo 3.2 & 3.3. These under girder inspection vehicles cannot move to the side spans. Directly under the orthotropic steel deck of the bridge, two inside girder inspection vehicles are installed as shown in Fig.3.2 and Photo 3.4. Both end sections of the floor can be extended and contracted. When the vehicle is moving under the bridge deck, the end sections need to be contracted to avoid the collision against the inclined truss members. When the vehicle stops, the end sections can be extended so that the outer surface of the truss girder can be inspected[2)]. The truss girder of the Matadi Bridge is continuous and the inside girder inspection vehicles move through the truss girder. Hence these vehicles can move from one anchorage to the other easily. One cable inspection vehicle was prepared at the time of the completion of the bridge. The vehicle is shown in Fig.3.4 and Photo 3.5. There is one pair of tower inspection vehicles which are suspended from the tower top. The vehicles are shown in Fig.3.5 & Photo 3.6.

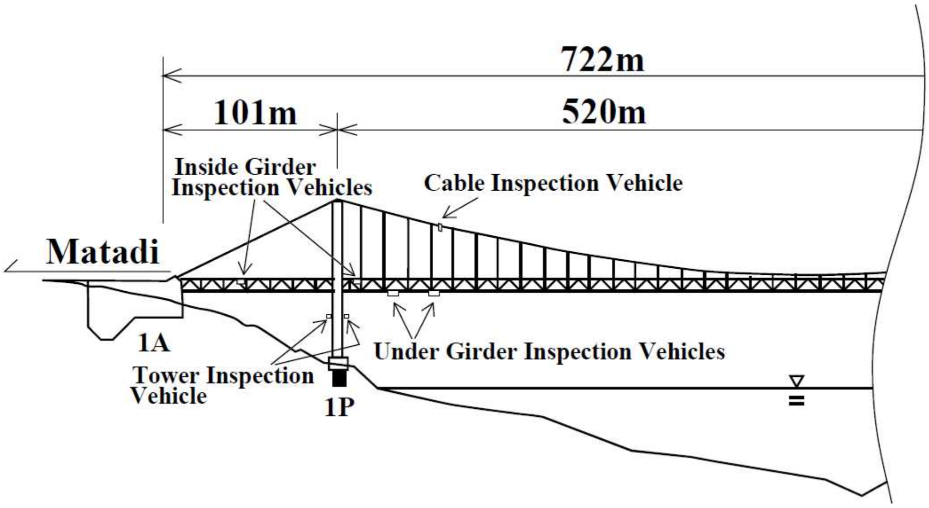
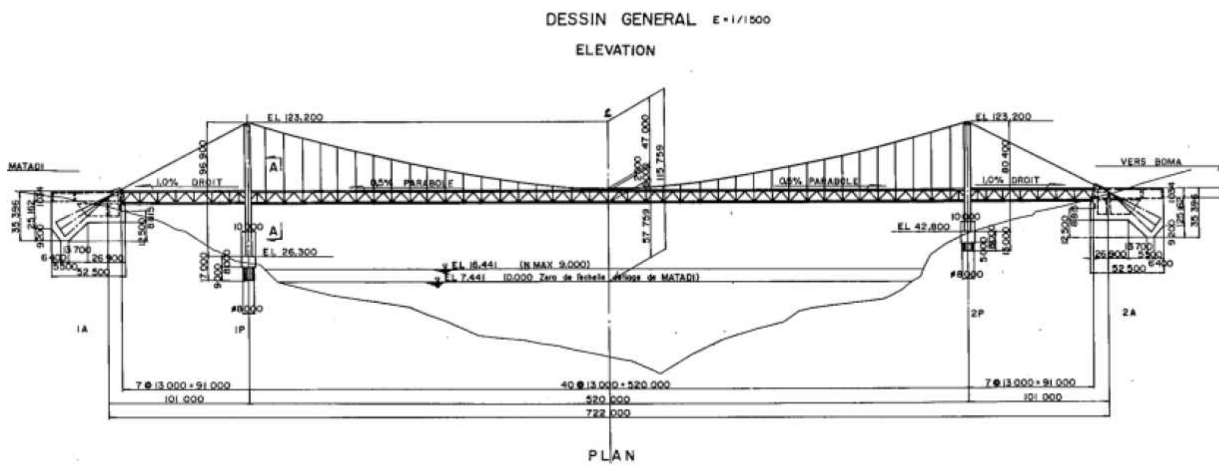
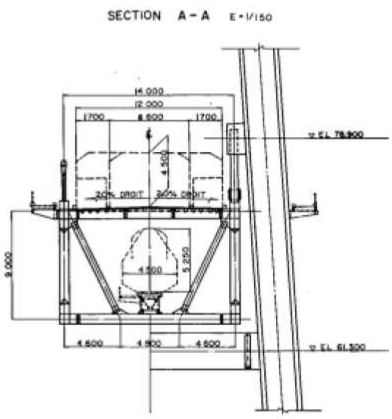
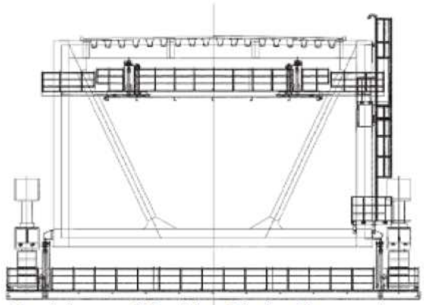


Fig.3.2 Inspection Vehicles of Matadi Bridge







Positions of Inside Girder Inspection Vehicle, Under Girder Inspection Vehicle and Vertical Ladder

Fig.3.3 General Drawing of Matadi Bridge and Positions of Vehicles (Source: Drawings of Matadi Bridge)

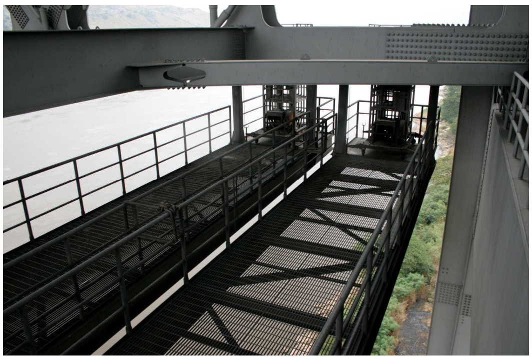


Photo 3.2 Two Under Girder Inspection Vehicles

Photo 3.4 Inside Girder Inspection Vehicle

Photo 3.3 Elevating Platform on Inspection Vehicle

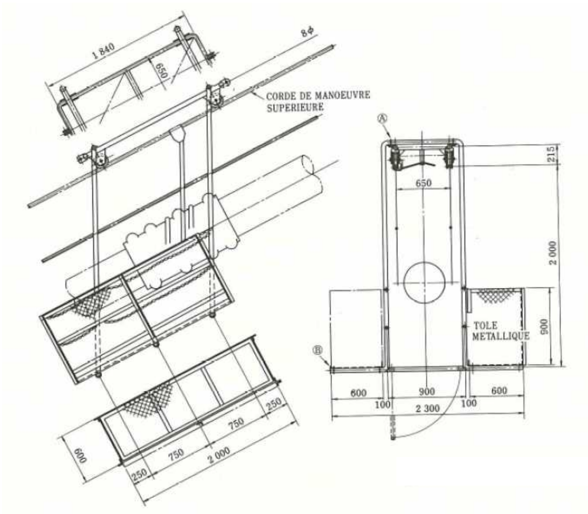


Fig.3.4 Cable Inspection Vehicle

Photo 3.5 Cable Inspection Vehicle manufactured in Japan

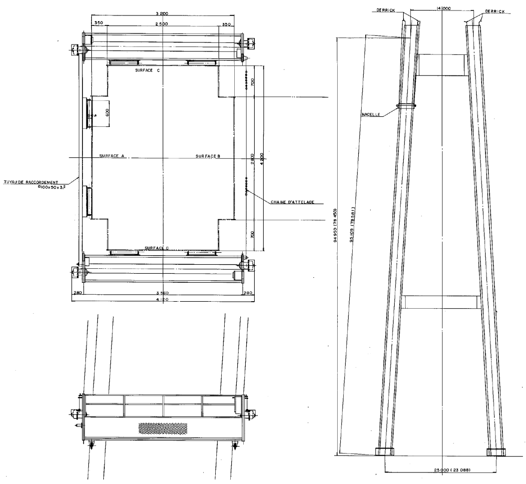


Fig.3.5 Tower Inspection Vehicles

Photo 3.6 Tower Inspection Vehicles

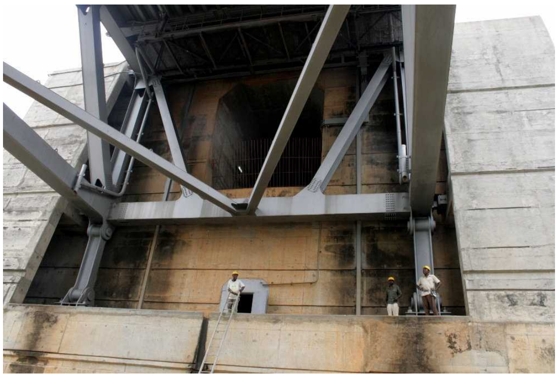


Photo 3.7 End link of Matadi Bridge

The structure form of the Matadi Bridge shown in Fig.3.1 is similar to that of the Shimotsui-Seto Bridge shown in Fig.2.30. The side spans of both bridges are not suspended by the hanger cables. For the Shimotsui-Seto Bridge, the noise problem of the end link happened as introduced in Chapter 2. But this kind of problem did not happen to the Matadi Bridge. This may be because of the following facts. The reaction force of the end link of the Matadi Bridge shown in Photo 3.7 is much smaller than that of the Shimotsui-Seto Bridge. At present no train is running on the Matadi Bridge. Even in the future, the possibility of train services is very low because recently various kind of minerals are transported to the east side of African continent and exported. When the bridge was planned, the various kind of minerals were supposed to be transported from Kinshasa via Matadi to Banana port on the west coast of African continent. The noise problem happened when the heavier freight trains ran on the Shimotsui-Seto Bridge adopting the complicated semispherical boss and cover structure as shown in Fig.2.31. The end link of the Matadi Bridge adopted the simple pin connection as shown in the photo 3.7.

The problems of suspension bridges are, main cable corrosions, hanger rope corrosions, hanger rope vibrations, noise and structure problems of expansion joints, rusts of steel parts, humidity of anchorages, etc. Therefore these places need to be inspected closely and regularly using the inspection vehicles and other facilities.

For the case of the cable-stayed bridges, the problems are, vibrations of stay cables and consequent damages to the dampers or vibration suppressor ropes, noise and structure problems of expansion joints, rusts of steel parts, etc. These places need to be inspected in the same way as the suspension bridges.

For these activities, the inspection vehicles and the maintenance manual are very important. The manual should be prepared and the contents should be reviewed regularly based on the inspection results.

According to the 30 years maintenance works, the following requests were made from OEBK.

(1) The inspection vehicles on the side spans are requested.

(2) At the center span, there are a few places with rail level differences and it is difficult for the under girder inspection vehicles to pass through. It is requested to repair these differences. Some hand driving gears of the inspection vehicles are also broken and the repair of them are requested. On both ends of the under girder inspection vehicles, manual elevating platforms are equipped. Due to the deterioration over time, these platforms do not function properly. It is requested to repair or renew them.

(3) One of the hand driving gears of the inside girder inspection vehicles is inclined and the repair is requested. At one place of the rail of the inspection vehicle, there is about 2mm rail level difference and it is difficult for the vehicles to pass over this difference. There is one more similar place, too. These differences need to be repaired.

(4) The tower inspection vehicle was functioning well until 2010. But when the vehicle was left on the tower pier, a control panel was stolen. Since then the vehicle could not be used. New vehicles are requested.

(5) The expansion joints need to be maintained regularly but there are no access scaffoldings. New scaffoldings are requested.

(6) There are one truck with crane and one aerial working platform truck. They are very useful to supplement the inspection works and the repair works. As they have been used for nearly 30 years, they are already deteriorated considerably and the replacement is requested.

This chapter discusses the improvement activities for the problems stated above.

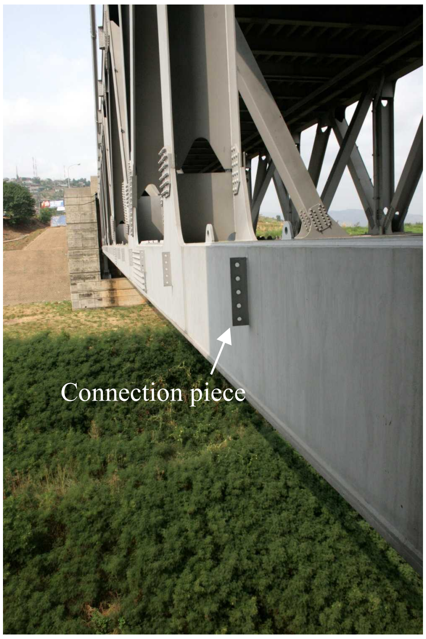


Photo 3.8 Side Span of Matadi Bridge

Photo 3.9 Repainting of Side Span, (Source; OEBK)

Photo 3.10 Driving gear running on rail

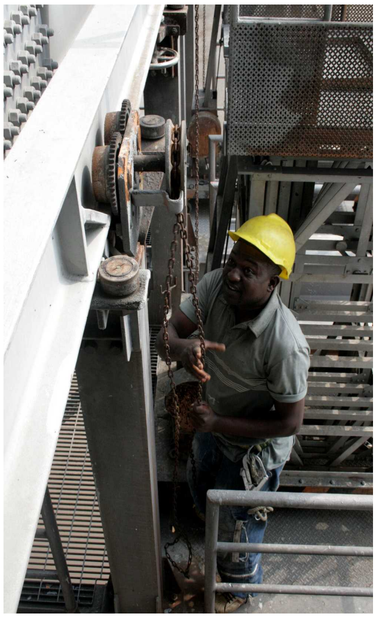


Photo 3.11 Driving vehicle

3.3 UNDERSTANDING OF PROBLEMS AND SOLUTION PLAN

In this section, the problems and the countermeasures for these problems are assessed in detail.

3.3.1 Under Girder Inspection Vehicle

No under girder inspection vehicles are equipped on the side spans from the beginning, as shown in Photo 3.8. But the connection pieces for the brackets to fix rails exist. The side span vehicles may have been omitted to reduce the initial cost with the reason that the side span could be maintained from the land[2)]. As shown in Fig.3.3, the side spans are over the land but it is very high from the ground. At the center span, there are two inspection vehicles and OEBK utilizes them for the daily maintenance works and used them for the repainting works from 2003 to 2009[1)]. On the side spans, there are no vehicles and the girder was repainted in a manner

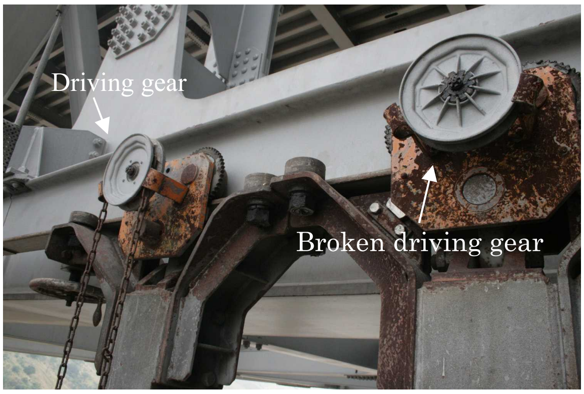


Photo 3.12 Healthy driving gear, left and gear without chain, right

Photo 3.13 Rail bent

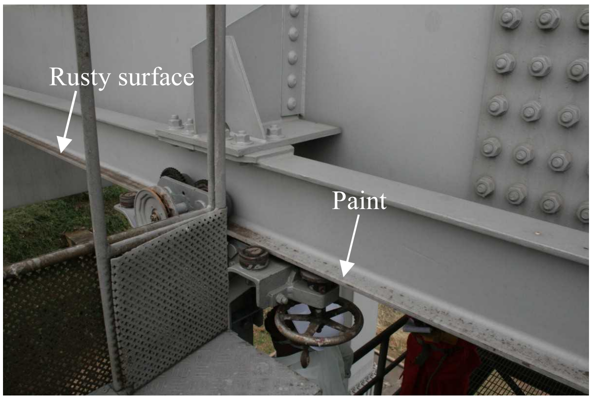


Photo 3.14 Rail with no friction on right, paint remains and rail with some friction on left, rusty rail surface.

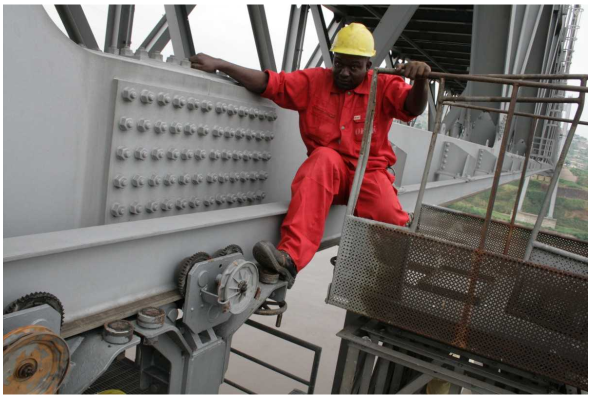


Photo 3.15 Vehicle pushed by Technician holding lower truss chord.

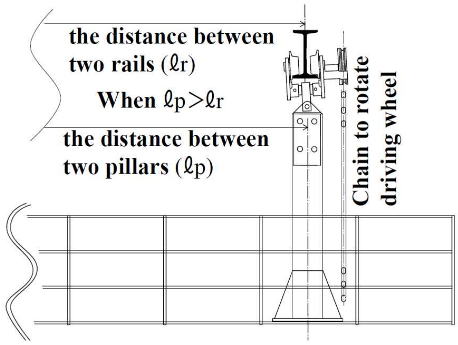


Fig.3.6 Inclination of driving gear due to difference of two distances, ℓr & ℓp

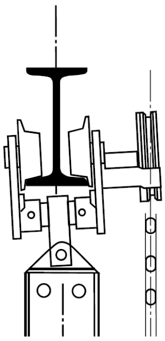


Fig.3.7 Partial Enlargement of Fig.3.6 & Detachment of driving wheel from rail

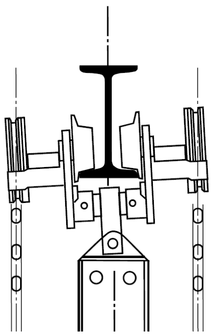


Fig.3.8 Improvement of driving gear, both sided driving wheels

shown in Photo 3.9. As this work becomes much safer with the side span inspection vehicles, OEBK strongly requests the installation of vehicles on the side spans. Another problem of the center span inspection vehicles is as follows; One inspection vehicle is supported by four pillars hung from the rails fixed on the truss girder. (Photo 3.2 & 3.10) Between pillars and rails, hand driving gears are installed and the vehicle can be moved manually. Moving the vehicle requires considerable labor, as shown in Photo 3.11. Some driving gears are already broken after about 30 years of service (Photo 3.12). Therefore these driving gears need to be replaced by new ones. Another problem is that as the rail is not straight (Photo 3.13), wheels of driving gears sometimes detach from the rail and the vehicle cannot move. This can be confirmed from the rail surface. As shown in Photo 3.14, a painted surface remains on the rail where the driving wheel detaches and no friction occurs. But with some friction, the paint is removed and the rusty rail surface appears as shown on the left of the rail in Photo 3.14. When the friction is weak and the inspection vehicle cannot be moved, a technician will push the vehicle as shown in Photo 3.15. It is confirmed that there are at least three places where this happens. This phenomenon can be explained as follows; The width of vehicle has a fixed length. But the distance between two rails vary as shown in Photo 3.13. If the distance becomes narrower, the driving gear becomes inclined as there is a pin below the gear to allow the rotation. Although this inclination may be small, the friction between the rail and the wheel may become weaker and the wheel may run idle. This situation is explained in Fig.3.6 and 3.7. Another possibility is that even though two distances are the same $\ell r = \ell p$, if rails are fixed in the inclined position, the same situation will appear. If both wheels of the driving gear can be driven, then this problem can be solved. (Fig.3.8) The possibility of the improvement of both sided driving wheels was inquired to the driving gear maker. The answer was that it was possible although a special order was needed. It was decided to adopt this improvement.

OEBK requests to electrify the inspection vehicles because it is laborious to drive the vehicles manually and also they know the electrified inspection vehicles of Japan, but unfortunately this request is rejected because it is difficult to maintain electrified machines[3)] and it is better to avoid the theft.

Another problem is the level difference of the rails as shown in Photo 3.16. The difference is small and about 2mm or less. If the vehicles are electrified, this difference is not a problem. But the vehicle is manually driven, sometimes it is stopped at this difference. Then technicians drive back the vehicle and try to pass through the difference with the momentum. There are at least three places of this difference. Then it is decided to grind the surface to erase the difference.

The last problem is about the elevating platforms placed on both ends of the under girder inspection vehicle. (Photo 3.3) According to OEBK, this platform can be elevated but cannot be stopped properly and sometimes the platform slips downward. As these platforms are too old and exceed the service life, it is decided to replace them by new equivalent ones.

From the above experiences, the feedback for the design is as follows; If a girder is too high from the ground, the under girder inspection vehicle is needed. A small rail bent shown in Photo 3.13 is to some extent inevitable so that the adoption of the both sided manual driving gears are recommended. The rail level differences should be erased from the beginning when the manual driving inspection vehicles are adopted.

3.3.2 INSIDE GIRDER INSPECTION VEHICLE

The inside girder inspection vehicle is shown in Photo 3.4. The problems of the inside girder inspection vehicles are as follows; The driving gears of these inside girder inspection vehicles are already worn out in the same way as those of the under girder inspection vehicles. One of them is even inclined as shown in Photo 3.17. These driving gears need to be replaced by new ones. For the inside inspection vehicles, the problem of running idle of driving wheels does not exist. Another problem is the rail level difference shown in Photo 3.18, which is the same problem as that found on the under girder inspection vehicles. Two places of such difference are found. To solve the problem, the differences will be ground, too.

On the side spans, there is 1% longitudinal gradient as shown in Fig.3.3. When the inside girder inspection vehicle climbs this 1% slope, technicians utilize the chains to rotate the driving wheels and this is a laborious work. (Photo 3.19) But when the vehicle descends down this 1% slope, technicians push forward the vehicle by

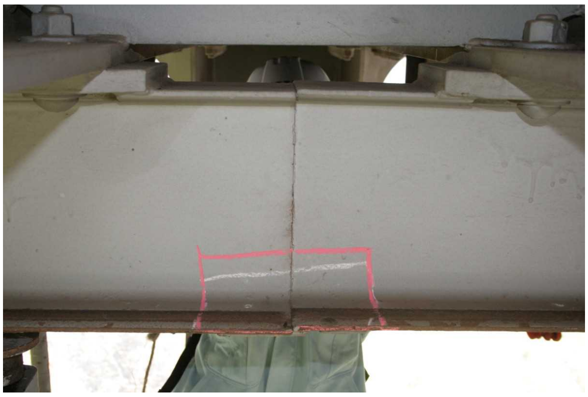


Photo 3.16 Rail level difference

Photo 3.17 Inclination of driving gear

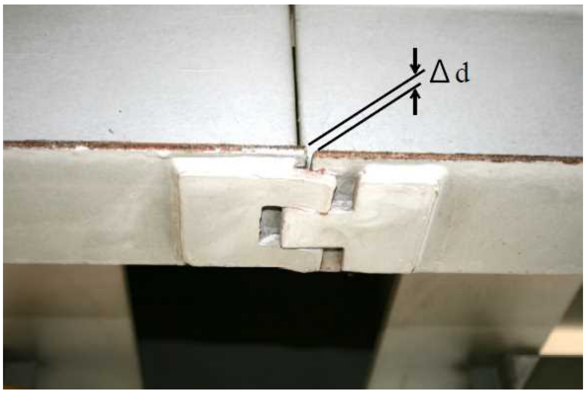


Photo 3.18 Rail level difference

Photo 3.19 Driving vehicle by chains

Photo 3.20 Pushing vehicle

Photo 3.21 Climbing route

holding the steel truss members. (Photo 3.20) It is much faster and easier to push the vehicle by this manner than to rotate the driving wheels by chains, because of the gravity. When engineers and technicians of OEBK inspect the bridge by this vehicle from the tower to the anchorage direction and the work ending time comes, they stop the vehicle near the anchorage, extend the vehicle floor to its maximum and climb up to the bridge deck level as shown in Photo 3.21 with the help of the connection piece of the truss chord and their safety belts. This is because it is too laborious to drive back the vehicle to the tower position, where there is a ladder. As this may be a dangerous practice, it was proposed to install a new ladder near the anchorage, which is the same type of ladder shown in Photo 3.22. At the center span, there are three ladders at every quarter span length, i.e. 1/4 point, 1/2 point and 3/4 point[2)].

A manually driven geared trolley is a device which lifts up a heavy material and transports it laterally along a rail installed above the trolley. The geared trolley consists of two parts, one is the lifting part and the other is the laterally moving part. This laterally moving part is adopted for the driving gears of the inspection vehicles of the Matadi Bridge. According to the maker of geared trolleys, manually driven geared trolleys need to be planned on horizontally installed rails. It is feared that if the gradient is larger, the geared trollies move back to the lower position. The maximum longitudinal gradient of the Matadi Bridge is 1%, as this bridge is a railway/roadway combined bridge and trains cannot climb steep slopes, the maximum gradient is small compared to the road only bridges which have larger gradient, for example the Akashi Kaikyo Bridge has a maximum gradient of 3% and the Suez Canal Bridge, 3.3%. If a bridge is planned with larger longitudinal gradient, the electrified inspection vehicles need to be planned. In developing countries, electric motors or generators/batteries are easily stolen and the countermeasures against the theft need to be carefully planned. One of the solutions is to plan the electric parts, such as motors, generators/batteries, etc. detachable. Then it is possible to bring back these electric parts to the storage after the bridge inspection works. Another solution is to arrange several security guards to prevent the theft of electric parts.

From the above experiences, the feedback for the design is as follows; The proper arrangement of facilities, such as ladders, to get on and off manually driving inspection vehicles, is needed. The rail level differences should be erased from the beginning in the same manner as the case of the under girder inspection vehicles. If the girder longitudinal gradient exceeds more than 1%, the electrified inspection vehicles need to be planned.

3.3.3 TOWER INSPECTION VEHICLE

The tower inspection vehicles are utilized for the inspection of tower surfaces and the repainting works. They were working well until 2011, when the control panel was stolen. The tower inspection vehicle was left overnight on the top of the tower pier which was about 10m high from the ground and the access to this place was very difficult. The commercially available suspended working platforms for the window cleaning of tall buildings were adopted for the original tower inspection vehicles of the Matadi Bridge. The platforms with the equivalent performance were available even at present, so that it was decided to buy new ones.

3.3.4 CABLE INSPECTION VEHICLE

The cable inspection vehicle is utilized to inspect cables, cable bands and cable band bolts. Especially to inspect the underside of cables. The upside of cables can be inspected by walking on the cable surface. But the underside can be inspected only from the inspection vehicle. This vehicle is utilized for the repainting works, too. When the bridge was completed, one cable inspection vehicle was prepared. (Photo 3.5) But OEBK judged that one inspection vehicle was inadequate and they asked one institute in Matadi to make one more cable inspection vehicle after the Japanese vehicle model, but with the improvement of the slightly lower platform to facilitate the painting of the cable bottoms. (Photo 3.23) This improvement should be incorporated for the design of future cable inspection vehicles. Also the fact that OEBK increased the number of inspection vehicles by themselves suggests that at least two cable inspection vehicles are needed for the maintenance of the suspension bridges.

3.3.5 SCAFFOLDING FOR EXPANSION JOINT

The finger expansion joints are installed between the anchorages and the ends of the truss girder. Under the expansion joint, there are several supporting rails. One problem is that some supporting rails are moving out regularly so that OEBK is continuously monitoring these rails and pushes back these rails. (Photo 3.24) To monitor and to push back rails, a scaffolding is indispensable. They manage to reach the place directly under the expansion joint from the inside girder inspection vehicle. The permanent scaffolding is needed. The rails are fixed on the abutment with the structure shown in Fig.3.9. The rubber plates under the rails have deteriorated and the rails may not be fixed firmly. This may be the reason why the rails keep moving out. As it is not easy to change the structure to fix the rails more firmly and OEBK always inspects the condition and pushes back the rails regularly, there are no serious problems so far. Therefore it is judged sufficient to add the scaffolding only.

When the permanent scaffolding under the expansion joint was installed, the concrete surface of the anchorage, on which the scaffolding was planned to be installed, was inspected. The rock pockets and the repaired surfaces were found and it was judged not to install the permanent scaffolding on this surface. Then it was judged that it was better to install a temporary scaffolding with the steel pipes, which could last for a considerably long time.

As the expansion joint is one of the most important parts which need to be inspected regularly, there should be some device to facilitate the inspection from the beginning. One of the method may be to extend the rails of the inside girder inspection vehicles slightly beyond the truss girder end. The rails are shown in Photos 3.17 and 3.19. The rails needs to be extended slightly to avoid the collision against the anchorage surface. By this extension, the inside girder inspection vehicle can move slightly closer to the anchorage and the expansion joint would have been inspected more easily.

Photo 3.22 Center Span Ladder

Photo 3.23 Cable inspection vehicle manufactured by OEBK

Photo 3.24 Rail Moving of Expansion Joint

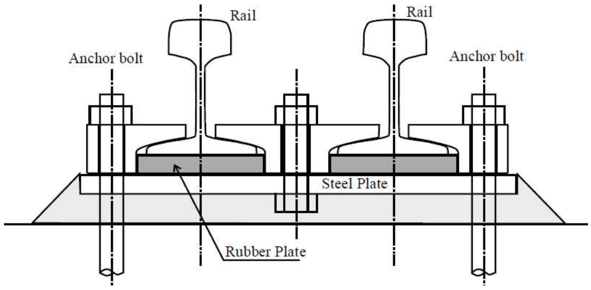


Fig.3.9 Rails and their fixing Structure, Rubber Plate under Rail

Photo 3.25 4t Truck with Crane

Photo 3.26 Aerial Working Platform Truck

Photo 3.27 10t Truck Crane

Photo 3.28 Backhoe Loader

3.3.6 TRUCK AND AERIAL WORKING PLATFORM

OEBK had one 4t truck with crane, one aerial working platform truck, one 10t truck crane and one backhoe loader. 4t Truck is shown in Photo 3.25. This truck had run already 1,130,000km (far more than 100,000km) and was worn out. This needed to be renewed. The aerial working platform truck is shown in Photo 3.26. This had a performance of 200kgX24m. This truck was used for the exchange of the electric bulbs of lighting poles and the inspection of the main cables. This truck had run more than 730,000 km. The platform could be extended but it could not be rotated because of the malfunction. This truck also needed to be replaced by an equivalent machine.

The 10t truck crane is shown in Photo 3.27. This is a large truck crane and used for the time of the construction. Therefore for the maintenance activities, this size of large truck crane might not be needed. OEBK used this truck crane to load the cable inspection vehicles on the cables. The outriggers of this crane did not work properly due to the deterioration.

The backhoe loader is shown in Photo 3.28. This loader was used for the maintenance of road sections. When this Photo 3.28 was taken, this machine was out of order because of the deterioration of the hydraulic packings. But later this machine was repaired by OEBK and they utilized this machine. This machine is very useful for the maintenance of earthwork sections. But this might not be needed for the maintenance of bridge proper and it was decided not to renew. This is a product of Japanese company. In Kinshasa there is a branch of one European Company which deals with the products of this company. There are many kinds of metal mines in DRC and many products of this company are utilized in DRC. According to this branch, they maintain even older machines than the backhoe loader of OEBK and they could repair the backhoe loader to the complete condition. This information was transmitted to OEBK and OEBK could repair the backhoe loader completely if they so wished.

As it was decided not to replace 10t truck crane and this 10t truck crane was no longer reliable due to the deterioration, the size of the truck with crane which could load the cable inspection vehicles on the cables needed to be decided. The loading of the cable inspection vehicles was investigated. At first, it was thought that the loading of vehicles on the cable was easy at the center of the center span or near the anchorages where the cables were the lowest. But the situation was a little complicated because there were lighting poles just beside the main cables and the inspection vehicles could not pass these poles. The situation is shown in Photo 3.29. The lighting pole is quite close to the main cable, the inspection vehicles needed to be placed beyond this pole. The pole is 8.0m high. As shown in Photo 3.30, the vehicle needed to be placed where the hand rope height was about 11.1m. The height and the weight of the vehicle manufactured in Japan is about 2.5m and 332kg respectively from the document of the construction. The height of the vehicle manufactured by OEBK is slightly higher and assumed to be about 3.0m. The weight can be assumed to be 350kg. From these data, the necessary capacity was as follows;

(1) Lifting Height 11.1m+3.0m+1.0=15.1m where 1m is the length of the sling.

(2) Lifting Weight 350kg

(3) Operating radius 5m

One example of a truck with crane which satisfies the above specification is ZE505L, a product of

Japanese truck crane maker. When 15.69m boom is used, the capacity of lifting weight is 530kg. The relationship of capacity of ZE505L and the cable inspection vehicle is shown in Fig.3.10. ZE505L has the capacity of 2.93t×3.9m=11.427tm. A truck with crane which has an equivalent capacity to ZE505L is needed.

Photo 3.29 Lighting pole and main cable

Photo 3.30 Place for installation of cable inspection vehicle

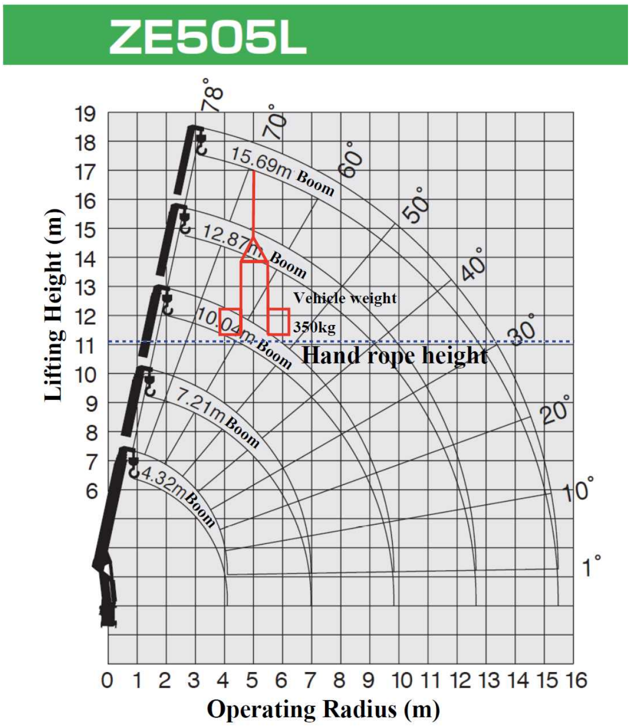


Fig.3.10 Crane Capacity & Cable Inspection Vehicle
(Source: modified from[4)])

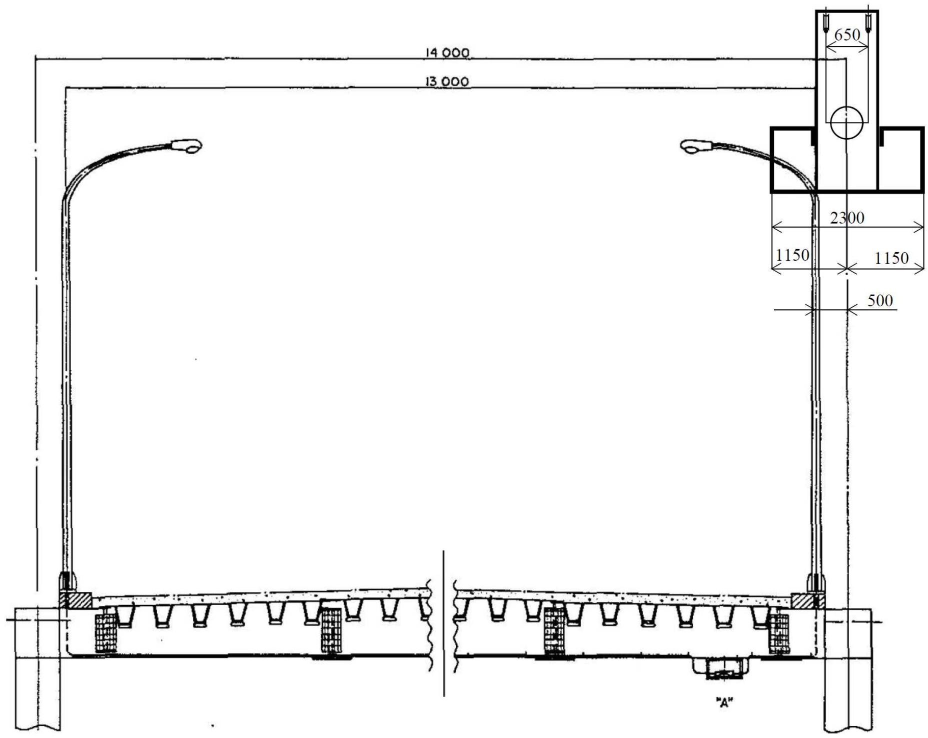


Fig.3.11 Lighting pole position on main girder and cable inspection vehicle position

In Japan, the road surfaces of long span bridges over the sea are lighted to secure the road traffic[5)]. The average luminance of the road surface is prescribed as 0.5cd/m^2 [6)]. To achieve this luminance, lighting poles are placed along the bridge deck. The Matadi Bridge must have adopted the same standard. In Fig.3.11, the positions of the lighting pole and the cable inspection vehicle are shown. Between the center of the upper truss chord and the center of the lighting pole, there is only 0.5m space. The center of the upper truss chord and the center of the main cable coincide each other. The width of the cable inspection vehicle is 2.3m, therefore more than 1.15m clearance from the center of the main cable is needed to move freely on the cable. But only less than 0.5m clearance exists and the cable inspection vehicles cannot move freely where the lighting poles height exceeds the cable height. To avoid this situation, the girder width needs to be widened about 0.8m for one side and in total, 1.6m. The whole structure of the bridge needs to be widened about 1.6m. This will push up the initial construction cost considerably. To achieve the lower initial construction cost, the narrower width girder might have been adopted for the Matadi Bridge, partly because a 10t truck crane which was used for the construction works could be left at the site to install the cable inspection vehicle on high positions of the main cable. But this slightly pushed up the cost of maintenance in the future, such as a truck with crane. Also the easiness of cable maintenance work was slightly lowered because the cable inspection vehicles could not move through the cable lower sections.

The dimensions of lighting poles of the Matadi Bridge, the Kurushima Kaikyo Bridges and the Suez Canal Bridge are shown in Table 3.2. From this table, lighting poles of about 8 to 11m height are needed for the long span bridges. When a new suspension bridge is planned in developing countries, the necessity of these lighting poles needs to be recognized at the design stage. If these poles are placed too close to the main cables, the cable inspection vehicles cannot pass through these poles, which results in a slightly higher maintenance cost. But the girder width becomes narrower and the initial construction cost becomes slightly cheaper. On the contrary to this situation, if the wider girder is adopted to allow the passage of cable inspection vehicles, the initial construction cost becomes slightly higher. In Japan, generally speaking, a wide girder width is adopted because the manpower cost for the maintenance in the future is higher.

Table 3.2 Lighting pole dimensions

Bridge name	Lighting pole height(m)	Lighting pole interval(m)
Matadi Bridge	8	26
Kurushima Kaikyo Bridges	10	30
Suez Canal Bridge	11	20

3.4 CONCLUSION

According to the improvement activities of the maintenance facilities of the Matadi Bridge, the following facts become clear.

(1) The Matadi Bridge is well maintained by OEBK, partly because the maintenance vehicles work well. This is partly because the vehicles are not electrified, the maintenance of these vehicles are comparatively easy and the possibility of theft of unelectrified parts is low. But after about 30 years from the completion, some defects and deterioration of the maintenance vehicles happened. As OEBK could not repair these deteriorations, Japanese experts were dispatched to repair and improve them at the site. If the Matadi Bridge were in Japan, each defect would have been repaired much earlier, soon after the detection. For the Matadi Bridge, 30 years of defects and the deteriorations were repaired at once, as the mechanical parts cannot be obtained in DRC. From this fact, in developing countries, durable maintenance vehicles which can last more than 30 years, are recommended. To achieve this, unelecrified maintenance vehicles should be investigated and adopted. In this case, the adoption of the both sided driving wheels is recommended to avoid running idle on the rails.

(2) The side span under girder inspection vehicles of the Maradi Bridge had not been prepared in the beginning with the reason that the side spans are above the ground. But the side span girders are very high above the ground. The side span under girder inspection vehicles should be prepared from the beginning.

(3) OEBK made one more cable inspection vehicle by themselves with the improvement of slightly lower platform to facility the inspection works of cable bottoms. This improvement should be incorporated for the future design of cable inspection vehicles. Also two inspection vehicles are needed to maintain one suspension bridge from the fact that OEBK made one more inspection vehicle by themselves.

(4) One of the biggest problems is the maintenance of trucks. There are no branches of Japanese Truck makers in DRC and the spare parts need to be procured from, for example, Republic of South Africa.

(5) In Sec.3.3.6, the selection of a truck with crane is discussed and a comparatively larger truck with crane was adopted. This is because the lighting poles are placed too close to the main cables. To avoid this situation,

the girder width should be widened from the beginning so that the cable inspection vehicles could pass through the lighting poles easily. But this will push up the initial construction cost. Therefore the bridge designers need to notice this fact and balance the initial construction cost and the future maintenance cost carefully.

(6) The Innoshima Bridge was completed in December 1983 and the Matadi Bride, May 1983. As discussed in Sec.2.8, the U-shaped under girder inspection vehicle was adopted for the Innoshima Bridge to facilitate the inspection of both sides of the truss girder. But for the Matadi Bridge, the flat type under girder inspection vehicle with the elevating platforms was adopted. The elevating platforms were the commercially available platforms. This flat type inspection vehicle was the one which was adopted by the Kanmon Bridge, a one generation older type. This might be because to reduce the initial construction cost. Another fact is that if the U-shaped under girder inspection vehicle is adopted, it is impossible to move the vehicle manually as the U-shaped vehicle is heavier than the flat type vehicle as shown in Fig.2.33. Then the electrification of the vehicle becomes necessary and consequently the theft prevention countermeasures becomes more important. Or if the electrified U-shaped under girder inspection vehicles had been adopted, the inspection vehicle would have been broken or vandalized much earlier and the maintenance work would have been much more difficult. In this sense, the adoption of manually driven flat type under girder inspection vehicle might have been quite correct decision.

(7) If a new suspension bridge or a cable-stayed bridge is planned in developing countries even now, where the possibility of theft of parts is high, the adoption of manually driven inspection vehicles is recommended. In this case a higher redundancy of manual driving gears are needed. The manually driving gears can be adopted for the maximum longitudinal gradient of up to 1%. If the gradient exceeds 1%, the manually driving gears may not be feasible. In this case, the electrified driving gears are needed. When the electrified driving gears are introduced, the careful countermeasures against the theft of electric parts is recommended.

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Chapter 4 CORROSION PROBLEM OF MAIN CABLES

4.1 INTRODUCTION

For the main cables of the Matadi bridge, the wrapping paste and the wrapping wire combination against the corrosion was adopted. In Japan, this combination was found out to be inadequate, and a dry air injection system was introduced. Therefore it was feared that the corrosion of this cable might be found. To examine the present condition, the main cables were unwrapped and investigated at the site. Some corrosion was found as expected. Then the countermeasures against the corrosion were planned.

4.2 MAIN CABLES OF MATADI BRIDGE

The Matadi Bridge, a suspension bridge in Democratic Republic of Congo (DRC hereinafter), was completed in May 1983 by the Japanese ODA. (Photo 3.1) For the main cables of the Matadi Bridge, ϕ 5.15mm wire was adopted. The main cables were constructed by the PWS method. One PWS strand consisted of 127 wires. The side span cable horizontal force balances at the top of the tower with the center span cable horizontal force. The side spans of this bridge are much shorter than the half of the center span and the tension force in the cable is larger, two extra strands are added on the side spans. The number of strands on the side spans is 56 and the cable diameter is ϕ 480mm. The number of strands of the center span is 54 and the cable diameter is ϕ 471mm[1).

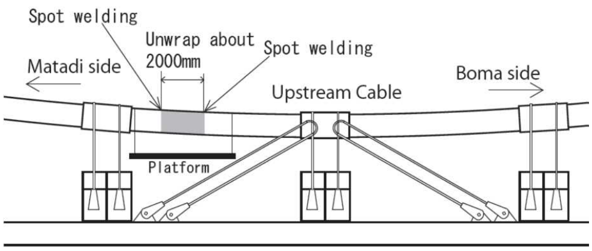


Fig.4.1 Position of opening inspection
(upstream position & downstream position are point symmetry.)

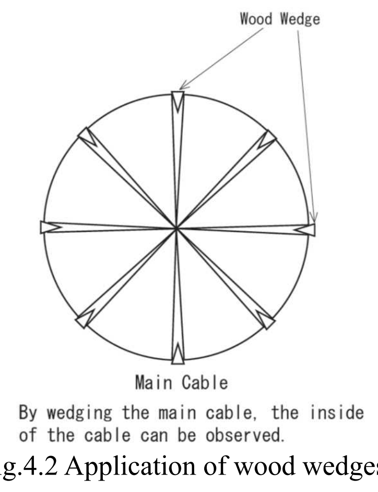


Fig.4.2 Application of wood wedges

For the main cables of the Matadi Bridge, the wrapping paste and the wrapping wire combination as a countermeasure against the corrosion was adopted. In Japan, this combination was found out to be inadequate, because after the examination of main cables of suspension bridges of the Honshu-Shikoku Bridges, some corrosion was found. Afterwards, a dry air injection system has been introduced to all of the main cables of the suspension bridges of the Honshu-Shikoku Bridges and most of other major suspension bridges in Japan. However, the cable protection system of the Matadi Bridge has never been changed.

From the experience of main cable corrosion in Japan, it was feared that the corrosion of the cable might be found and the new corrosion protection system might be needed for this bridge too. To examine the present condition, the main cables were examined at the site. This chapter reports this investigation and possible engineering feedbacks for maintenance are made.

4.3 BASIC STRATEGY OF INSPECTION

The center of the center span where the main cables are the lowest and it is the easiest to access, is chosen for the positions of the main cable inspection opening (Fig.4.1). The process is as follows.

(a) The scaffolding to facilitate the unwrapping and wrapping work are first installed under the main cables. Before unwrapping the cables, adjacent wrapping wires are spot-welded together to stop the unexpected unwrapping.

(b) A 2m section of each main cable, upstream and downstream, is unwrapped to inspect the inside of the cables. (The length of 2m is decided to facilitate the wedge driving into the main cable to inspect at least 6th or 7th layer of wires.)

(c) The wood wedges are applied on the cables from eight directions to inspect the inside (Fig.4.2). The depth of the wedges is about 5mm×7wires=35mm or more.

(d) After the inspection, epoxy resin zinc-rich paint is applied over the main cable wires. The main cables are wrapped by mild steel wires instead of the original wrapping wires. After the wrapping, the main cables are coated by the modified epoxy resin aluminum paint.

(e) Removal of scaffolding.

4.4 METHOD OF CABLE OPENING INSPECTION

4.4.1 WRAPPING SYSTEM OF MAIN CABLE

The wrapping system or the corrosion protection system of the Matadi Bridge is the same system as those adopted by the suspension bridges constructed up to 1990 in Japan, or those adopted in USA or UK. On the surface of cable wires, the wrapping paste is applied and over the paste, wrapping wires are placed tightly to keep the round shape of the cable. The surface of wrapping wires is coated by paint. This method is illustrated in Fig.4.3.

4.4.2 PROCESS OF INSPECTION

The process of inspection is as follows:

1) Installation of scaffoldings

→2) Inspection of cable surfaces

→3) Removal of wrapping wires

→4) Cable wire inspection immediately after the opening

→5) Cable wire inspection after the removal of paste

→6) Temporary closure (due to the bad weather when the rain water seepage into the cables is anticipated or due to wind more than 10m/s or due to the end of daylight)

→Return to 5) or proceed to 7)

→7) Wrapping and coating of main cable

→8) Removal of scaffoldings

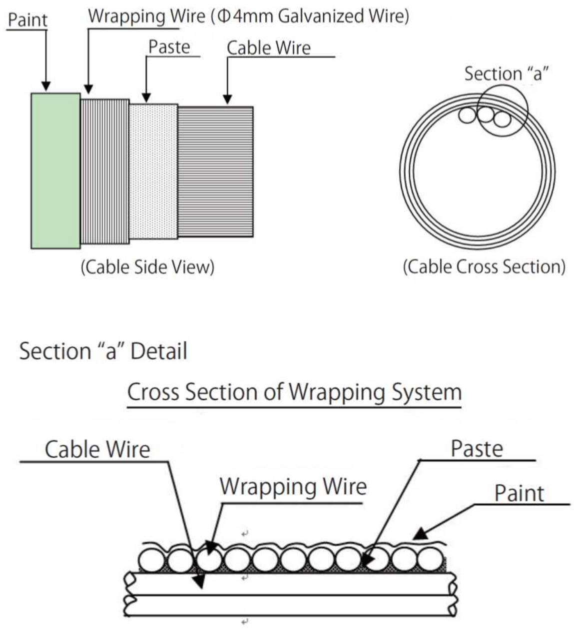


Fig.4.3 Main Cable Wrapping System

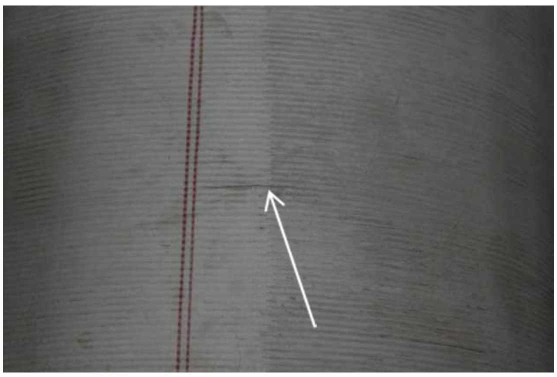


Photo 4.1 Paint Crack on Surface of Cable

4.5 RESULT OF INSPECTION

4.5.1 INSPECTION OF CABLE SURFACES

The whole lengths of main cables were visually inspected first. The condition of surface paint was very good because the whole surface was repainted about 10 years ago, during October 2003 to July 2004, by OEBK. After the inspection of cable surfaces, the paint adhesive test was conducted. The results showed a good adhesion remained and proved the good preparation work before the repainting.

However, some paint cracks were found. According to the investigation of the data given by OEBK, it was found that the adopted paint was not flexible paint. On the other hand, flexible polyurethane paint or flexible fluoro resin paint is adopted for a dry air injection system in Japan because they can prevent air leaking. If it is judged necessary to install a dry air injection system to the Matadi Bridge, the removal of the old paint and the repaint of the cable surface by the flexible paint to prevent air leaking, must be carried out beforehand. The calculated main cable surface area except cable bands is about 1000m^2, which is needed to calculate the amount of painting materials. One of the cracks is shown in Photo 4.1. One of the other problems was the missing caps of cable band bolts. The inside of the caps of many suspension bridges in Japan is threaded to prevent falling-off and at the same time to make the bands airtight. However, the caps of this bridge were not threaded so that some were missing. Also, some caps were damaged (Photo 4.2 & 4.3). If it is necessary to install a dry air injection system, then all of the caps need to be replaced by the threaded caps.

During the inspection, the retaining water at two cable bands was detected by OEBK. The fact that there were no other cable bands which contained water, was also confirmed by OEBK. One cable band was on the upstream cable and was the fifth cable band counted from the center band to Boma (a name of town, refer to Fig.4.1 and 4.9.) direction (Photo 4.4). Near the bottom of this band, there was a swollen point on the cable surface, when this swollen paint surface was opened, water drops came out. After a few days, the water marks on the deck turned to brown as shown in Photo 4.5, which showed the evidence of corrosion inside of the main cable. Photo 4.6 shows the corroded wires seen from the bottom of the band after the removal of sealant putty.

Photo 4.2 Inside of cap is not threaded

Photo 4.3 Damaged cap (Center)

Photo 4.4 Fifth cable band of upstream cable

Photo 4.5 Stains of rust water from main cable

Photo 4.6 Corroded wires seen from bottom of cable band

Photo 4.7 Fourth cable band of downstream cable

Photo 4.8 Corroded wires seen from bottom of cable band

It was suggested that OEBK keep the bottom open so that the water-retention would not occur again. The other cable band was on the downstream cable and was the fourth cable band (Photo 4.7) counted from the center band to Matadi direction. The condition of corrosion inside of this band was slightly better than that of the upstream band (Photo 4.8). To compare the results with the condition of cable bands without water-staying, the sealant putty of the upstream center band was removed and the inside was inspected. The result is shown in Photo 4.9. Only white rusted wires were observed inside of the cable band and the condition was much better.

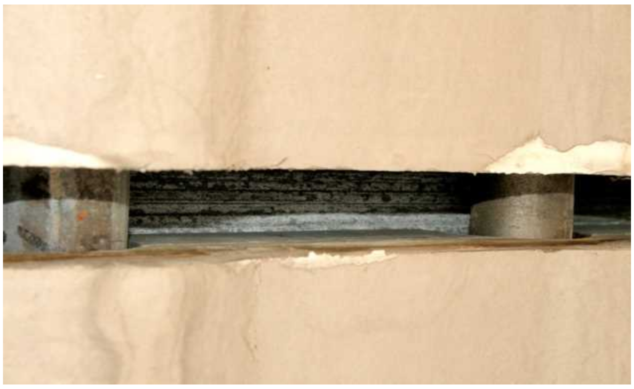


Photo 4.9 Cable wires inside of upstream center band seen from bottom

Photo 4.10 Wrapping wires are cut and removed.

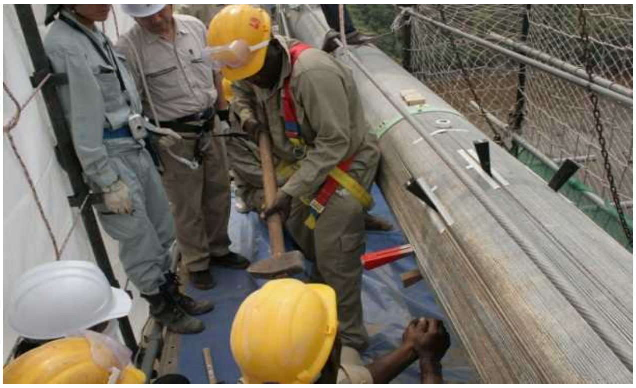


Photo 4.11 Application of wood wedges

4.5.2 UNWRAPPING OF MAIN CABLE

The main cable was unwrapped by cutting the wrapping wires as shown in Photo 4.10. Then the surface of wrapping paste was observed and the paste was removed completely to inspect the main cable surfaces and the wires. Then wood wedges were hammered into the main cable from the 8 directions as shown in Fig.4.2 and Photo 4.11 to inspect the inside of the main cable directly. The result of observation of the cable outer surfaces is shown in Fig.4.4. Surfaces were compared with the pictorial standard for the evaluation of cable strand (Outer surface) in Table 4.1. The compared result is shown in the center of Fig.4.4. Although Photos in Fig.4.4 show larger area of surfaces, the evaluated places are only at the center of the unwrapped section. As shown in Fig.4.4, the cable top surfaces were healthier than other sections.

Table 4.1 Pictorial Standard for Evaluation of deterioration of Cable Strands[2)] (Outer surface)

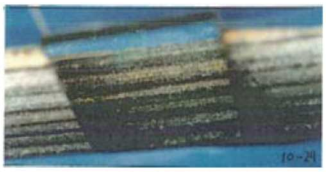
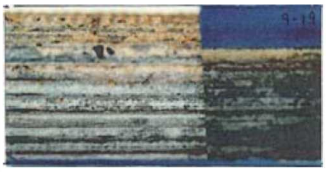
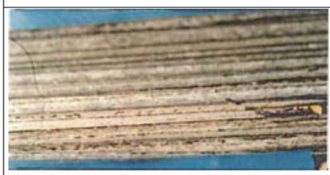
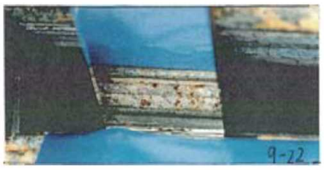
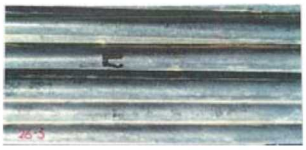
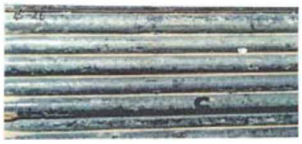
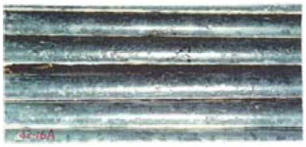
Standard Photographs	Criterion	Rating No.
	No deterioration	5
	Zinc white rust is observed.	4
	Zinc white rust is largely observed.	3
	Zinc white rust and steel spot red rust are observed.	2
		1

Table 4.2 Pictorial Standard for Evaluation of deterioration of Cable Strands[2)] (Inside)

Standard Photographs	Criterion	Rating No.
	No deterioration	5
	Zinc white rust is slightly observed.	4
	Loss of galvanized Zinc is slightly observed.	3
	Loss of galvanized Zinc is moderarely observed.	2
	Loss of galvanized Zinc is largely observed.	1

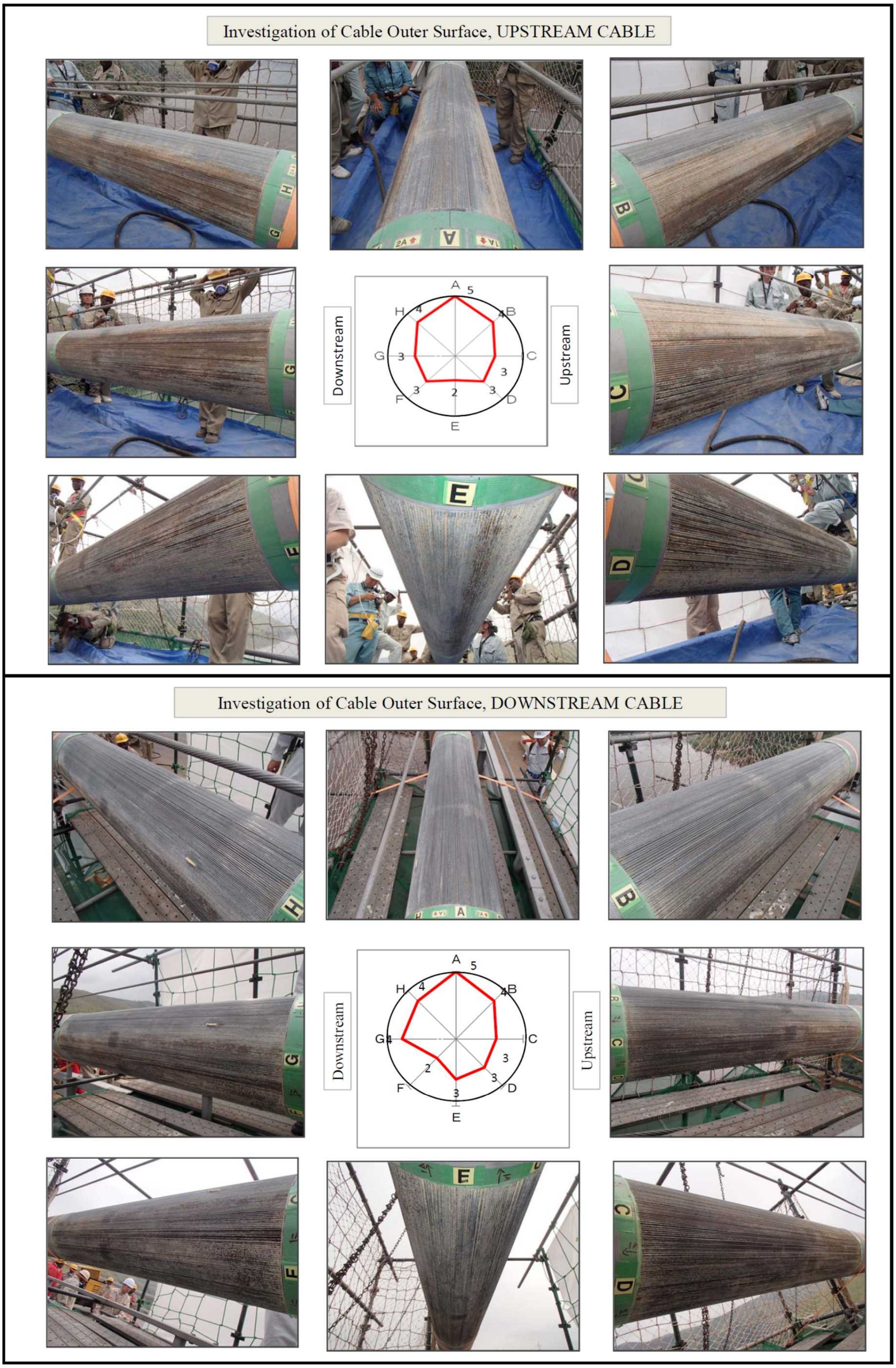


Fig.4.4 Investigation of Cable Outer Surface, Upstream and Downstream Cables

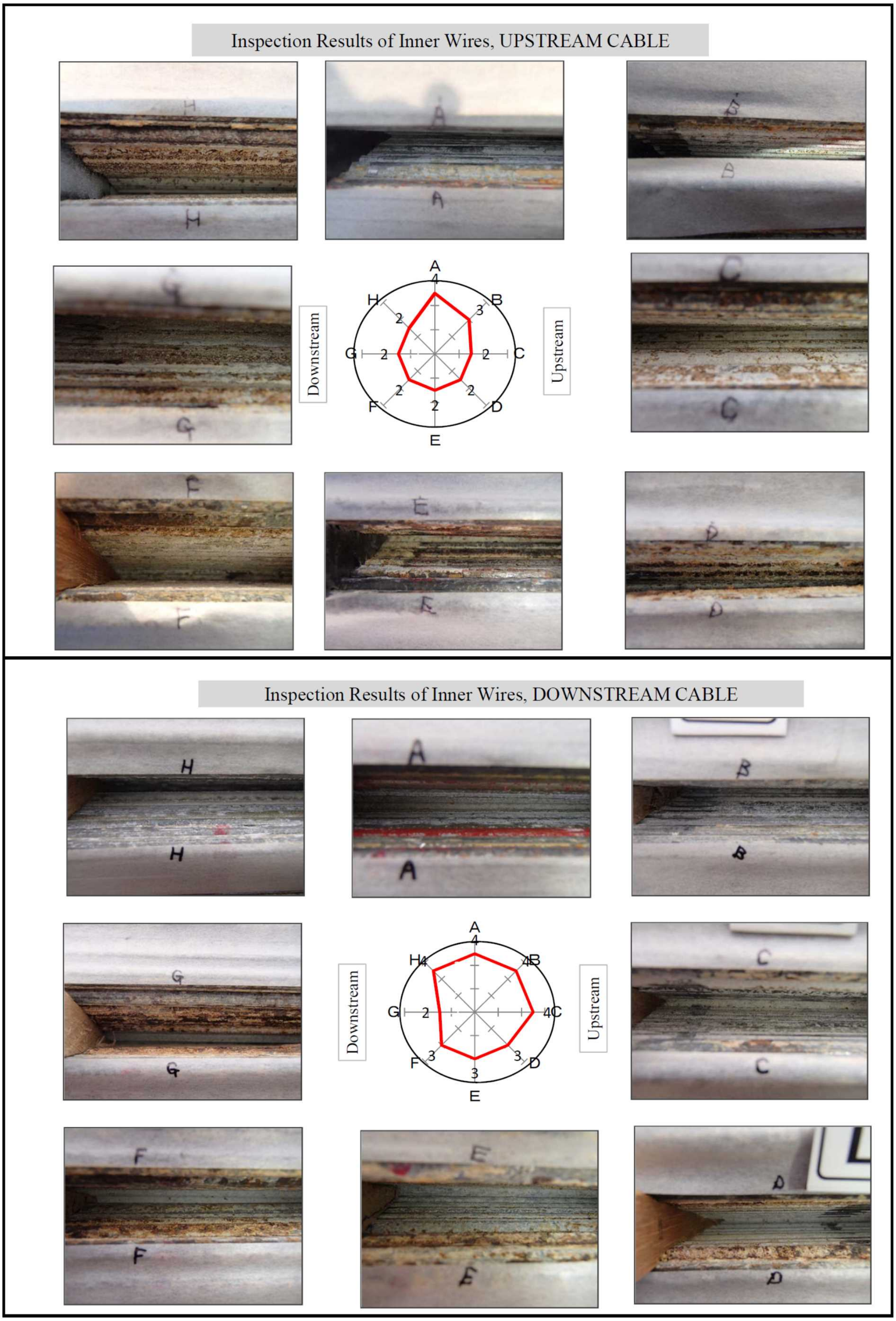


Fig.4.5 Inspection Results of inner wires, upstream & downstream cables


Photo 4.12 Main cable wrapping by plastic

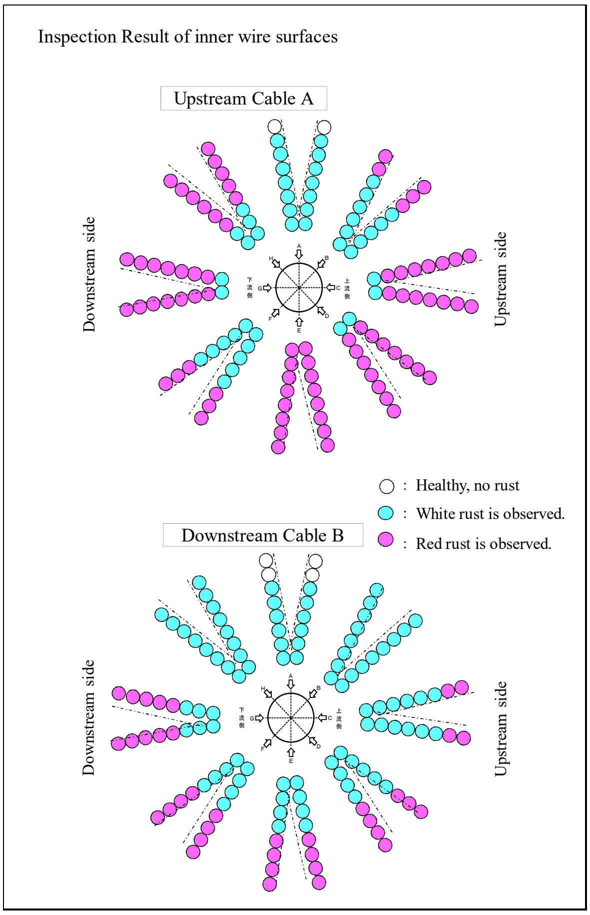


Fig.4.6 Schematic illustration of inside of cable

For the side surfaces and bottom surfaces of both upstream and downstream cables, red rust was clearly visible but the rust condition seemed to be better than the other suspension bridge cables in Japan, based on Japanese practices. From Fig.4.4, it is difficult to understand that the downstream cable is healthier than the upstream cable. But from the visual inspection, it was clearly visible that the downstream cable was less corroded. The result of observation of the inside of cables is shown in Fig.4.5 with the evaluation results at the center by the pictorial standard in Table 4.2. The condition of inside of cables was visually inspected and it was compared with the pictorial standard in Table 4.2, then the rating was decided based on the visual inspection. From Fig.4.5, only white rust is observed for the wires at the top of both upstream and downstream cables. For the downstream cable, only white rust is confirmed for the point of A,B,C and H. But for the upstream cable, white rust is observed only at point A. For other points of both cables, both white rust and red rust are confirmed. If the upstream cable and the downstream cable are compared, the downstream cable is healthier, because 2 ratings, the bad condition, can be found more on the upstream cable. The reason why the downstream cable was less corroded, was unknown. One of the possible reasons might be that there was a time difference for the cable wrapping works of the upstream cable and the downstream cable, and the upstream cable might have contained more rain water during the construction period.

Another fact was that no water was observed inside of the both cables probably because of the dry season (June 2012), which was different from the case of the Innoshima Bridge. Some water was confirmed inside of the cable for the Innoshima Bridge[4]. The condition of the inside of the cables is schematically illustrated in Fig.4.6. From this figure, the better condition of the downstream cable can be confirmed easily.

In Fig.4.6, the red spot rust is observed up to 8th layer of bottom side of the upstream cable. This is the worst rust of the upstream cable and for other places, the rust condition is not so serious. For the downstream cable, only up to 5th layer, the red spot rust is observed. From this observation, it can be said the condition of the cables of the Matadi Bridge is comparatively good, about 30 years after the completion.

For the Matadi Bridge, the white rust was observed up to 8th layers. The deeper layers were not investigated but it seemed that the white rust continued to the center of the cable.

The cable diameter of the Matadi Bridge is φ471mm at the center span and the wire diameter is φ5.15mm. Therefore there are about 90 wires in the cable diameter direction. Out of 90 wires, up to 8th layer of wires are partly red rusted. This condition seems to be better than main cables of other suspension bridges in Japan. The reason why the condition of cables of this bridge is comparatively good after about 30 years, may be due to the smaller temperature differences all around the year, although there are the dry season and the rainy season in DRC.

For the Forth Road Bridge in UK which was completed in 1964, the main cables were opened and inspected in 2004 and 2005, after about 40 years from the completion. To the surprise of the inspection team, fairly extensive corrosion and some wire breaks were found[3]. Compared to this bridge, the condition of the Matadi Bridge is better because no wire breaks were reported and the extensive corrosion was not found. This may be partly because of the climate differences between UK and DRC.

One observed phenomenon at the site was as follows. After the unwrapping of the main cable and after the one day work, to protect the main cable from the rain water, the cable was wrapped by plastic sheet in the evening.(Photo 4.12) The next morning, the condensation and the resulting water drops were not confirmed inside of the plastic sheet. On the contrary to this fact, the condensation and water drops are often observed in Japan. To examine this fact, the temperature and the relative humidity near the center of the downstream cable were investigated and the result is shown in Fig.4.7. This was the observed result of the outside air near the center band. As shown in Sec.2.2.4, the humidity of the cable ordinary section is continuously high, about 90% relative humidity, without being influenced by the outside air humidity and the humidity of the cable band section is closely related to the outside air humidity.

The saturation vapor pressure at T degree celcius (℃) can be obtained from WMO approximation formula.

$$\mathrm{e(T)} = \exp\left(19.5 - \frac{4303.4}{\mathrm{T} + 243.5}\right) \quad \mathrm{(hPa)} \tag{4.1}$$

From the measured data at the site of Fig.4.7, at 5:47 A.M., June 20th, 2012, the temperature and the relative humidity were, T=21.0℃ and Relative humidity=82.7%, which was the highest during this measurement. At this temperature, the saturation vapor pressure e(T) can be calculated and the value was,

$$\mathrm{e(T)} = 24.8 \ \mathrm{(hPa)}$$

As the relative humidity was 82.7%, the vapor pressure at that time was,

$$\mathrm{e} = 24.8 \times 0.827 = 20.5 \ \mathrm{(hPa)}$$

The temperature at which this vapor pressure becomes the saturation vapor pressure, is the dew point. This dew point can be calculated by reversing the above WMO approximation formula. The equation becomes,

$$T = \frac{4303.4}{19.5 - \ln(e)} - 243.5 = 17.9\ ℃ \tag{4.2}$$

Where e=20.5 (hPa).

This value of the dew point equals to the corresponding value of the relative humidity of 82.7% in Fig.4.7. The graph of Fig.4.7 was obtained directly from the hygrometer. Generally speaking the vapor pressure e= 20.5 hPa does not change largely in a short period, the dew point also does not change largely in a short period. Therefore the dew point is about 18℃ from June 19th to 22nd. This is the situation of the atmospheric air in the dry season.

As the next step, the situation inside of the cable is investigated. If it is assumed that the humidity of the inside of the cable was 90% based on the result of investigation of the Ohnaruto Bridge shown in Sec.2.2.4, then the vapor pressure inside of the cable at the temperature of T=21.0℃ was,

$$e = 24.8 \times 90\% = 22.3\ (\text{hPa})$$

The corresponding dew point of 22.3 hPa was,

$$T = \frac{4303.4}{19.5 - \ln(e)} - 243.5 = 19.3\ ℃ \tag{4.3}$$

This value is plotted in Fig.4.7. From Fig.4.7, the temperature at 5:47 A.M., June 20th, was 21.0℃, which was higher than the dew point inside of the cable, T=19.3℃. Then the dew inside of the plastic sheet was not formed. This might be the reason, why the dew condensation inside of the plastic sheet did not occur at the site. In this Fig.4.7, if the temperature would have lowered below T=19.3℃, the dew condensation would have had occurred inside of the cable only. If the temperature would have lowered further below T=17.9℃, then on both the outside and the inside of the plastic sheet, the dew condensation would have had occurred. The fact that the condensation and water drops are often observed in Japan, suggests that the temperature often lowers below the dew point of the cable inside, in this case, 19.3℃.

In Fig.4.7, 19.3℃ which corresponded to the dew point of the inside of the cable whose relative humidity was assumed to be 90%, was always higher than the dew point graph when the temperature was lower during the nights. Therefore no condensation was observed at the site. This may be one of the reasons why the deterioration of the Matadi Bridge proceeded slower than other bridges in the world.

This fact is further investigated. In Fig.4.8, the climate charts of Kinshasa and Takamatsu which is close to the Innoshima Bridge and other Seto Ohashi Bridges are shown. In the lower half of Fig.4.8, Maximum temperature, Average temperature and Minimum temperature of each city are shown. In the upper half, the relative humidity of each city is shown.

In Kinshasa, June to September is the dry season and the temperature is lower. The dew condensation often occurs when the temperature difference between the day time and the night time is large. In the dry season, this difference should be larger because of the radiative cooling in the night as the weather is dry and the sky is clear, than in the rainy season when the weather is cloudy. Although this difference is difficult to detect from Fig.4.8, as the data of each respective day is not shown. For the Matadi Bridge, even in the dry season, the dew condensation inside of the cable was not observed. Therefore it is highly possible that no condensation occurs all year round.

In Japan, the temperature difference between the day time and the night time may be larger than in Kinshasa, especially in autumn and in spring when the sky is clear and the radiation cooling occurs. This may be the reason why sometimes outside dew condensations are observed in Japan. As the relative humidity inside of the cables is reported to be about 90%, the dew condensation inside of the cable occurs more often than in outside air condition. This may be the reason why the condensation occurs inside of the cable and this phenomenon has been often observed during the cable opening inspection of the suspension bridges of the Honshu-Shikoku Bridge Expressway Company.

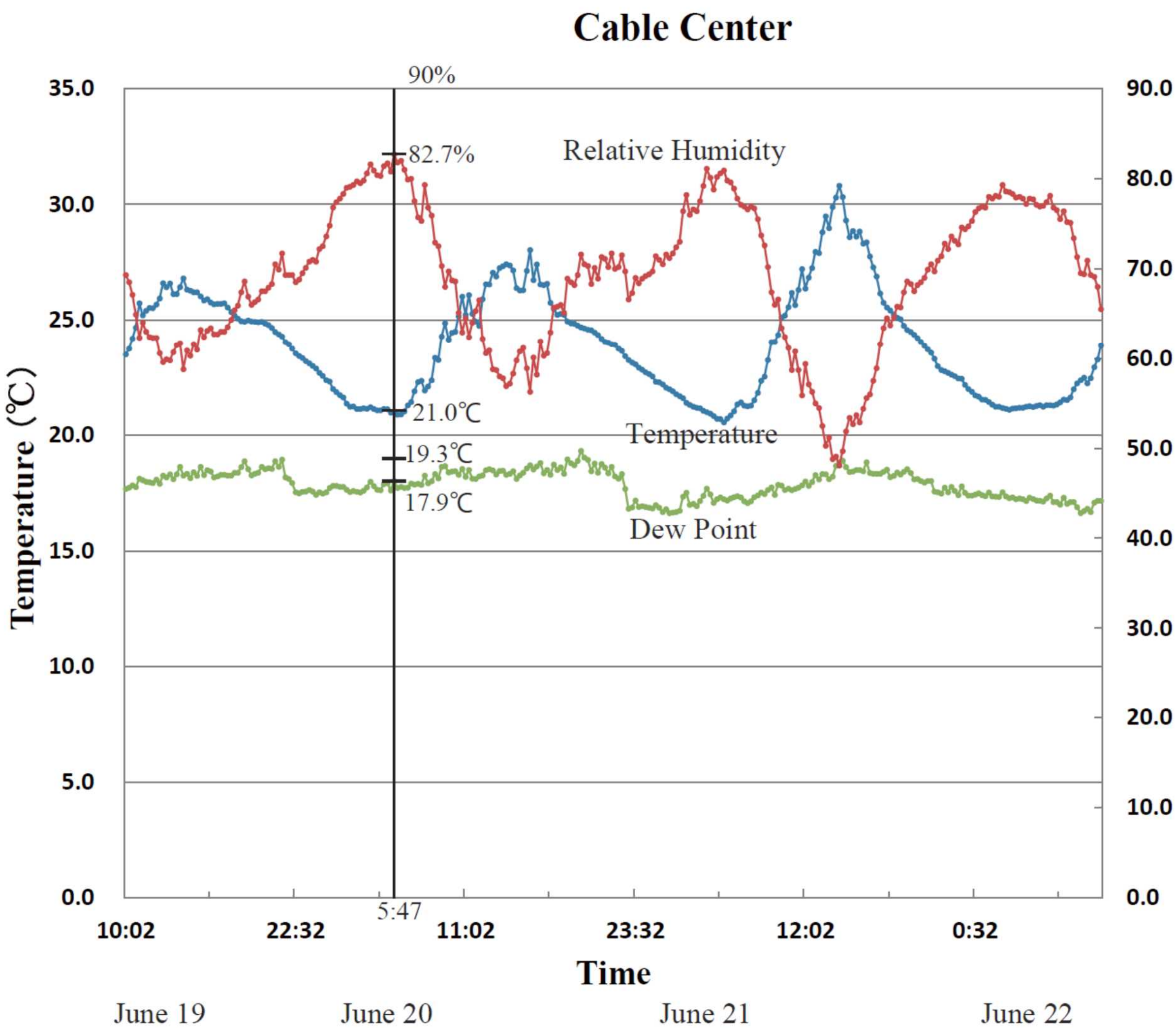
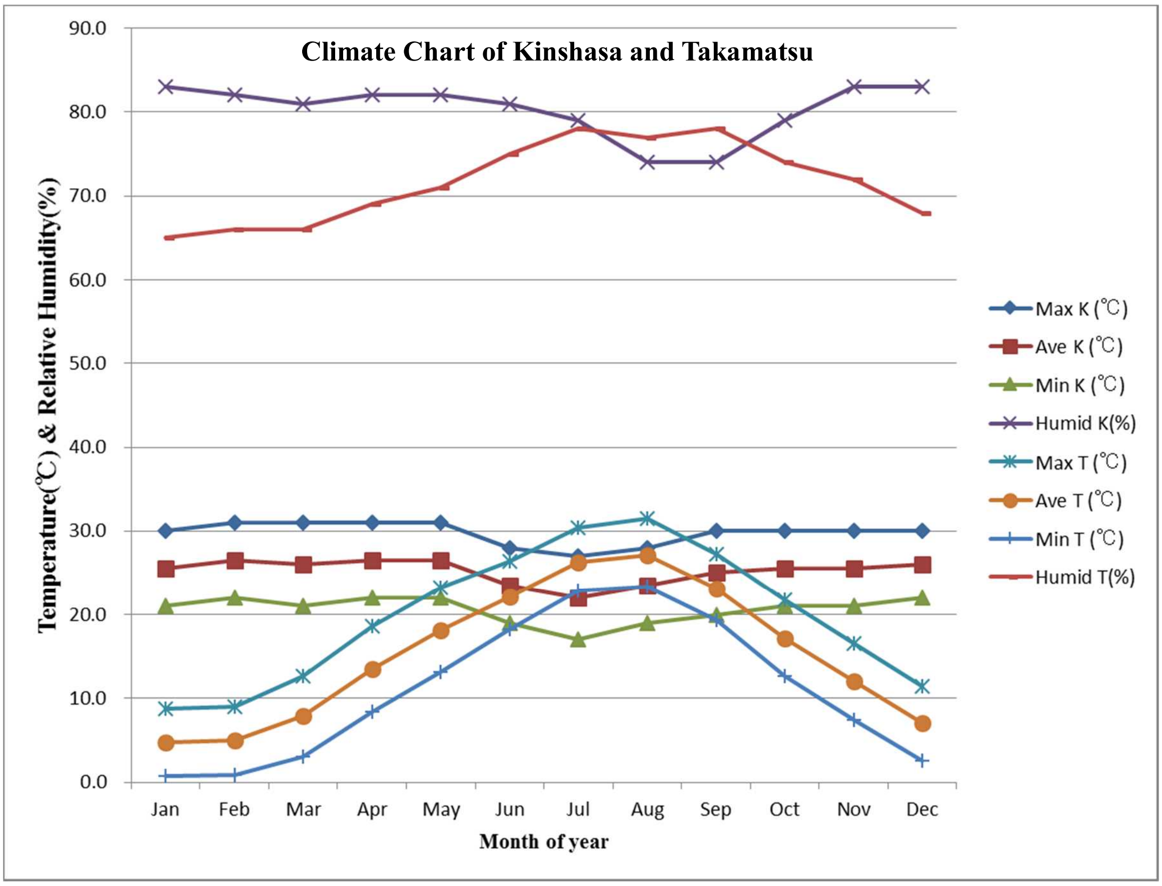


Fig.4.7 Temperature, Dew Point, Relative Humidity



K; Kinshasa(DRC), T: Takamatsu(JPN)
Fig.4.8 Climate Chart of Kinshasa and Takamatsu (Data source; climatemps.com)

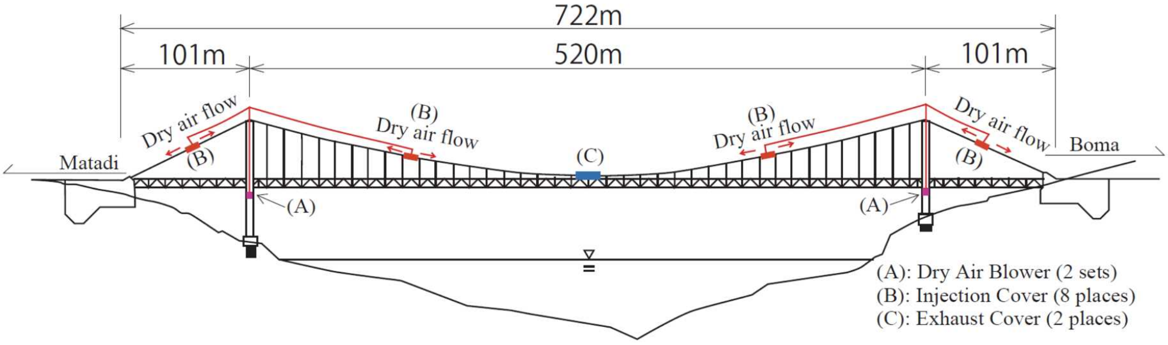


Fig.4.9 Possible dry air injection system for the Matadi Bridge

4.6 COUNTERMEASURE AGAINST CORROSION

About 30 years after the completion of the bridge, the main cables of the Matadi Bridge were first unwrapped and investigated. As a result of investigation, the main cables are found to be corroded to some extent. But the corroded condition was not so severe, probably due to the milder weather near the equator. But in the two cable bands, water-retention was detected. Therefore, near these two cable bands, the condition of wires of the cables could be expected to be worse. As the cables are continuous, only one point of deterioration can affect the total capacity of the cables. From this point of view, some countermeasures to stop the progress of corrosion are needed. In developing countries, a countermeasure which can be maintained easily, is preferable. A countermeasure without electrified machines may be a better solution. But the combination of wrapping wires and wrapping paste was already applied on the surface of the main cables of this bridge and it is very difficult to remove this combination. As discussed in Sec.2.2.5, it was practically almost impossible to protect the cables from the water by the conventional cable wrapping system only. Then the dry air injection system was developed. The same situation applied to this bridge, too. To remove the humidity inside of the cables, the only practical solution was the adoption of the already established dry air injection system. It was recommended to install the dry air injection system in the near future, which is already installed in almost all of the major suspension bridges in Japan.

For the design of new suspension bridges in developing countries, some other alternative methods can be applied. This will be discussed in Chapter 6.

Although the Matadi Bridge can accommodate 4 lanes of traffic, two pedestrian ways and one train rail in the future, but at that time only two lanes of traffic and two lanes of pedestrian ways are utilized, therefore there is a large room of load carrying capacity. Or the acting stresses of the main cables are far below the maximum capacity. But the cable wire breaks are irreversible process and if breaks are found, it is feared that the breaks will continue further. Therefore, it is strongly recommended to install the dry air injection system to prevent the future deterioration. The effectiveness of the dry air injection system is already confirmed by the suspension bridges of the Honshu-Shikoku Bridge Expressway Company[4]. One possible configuration of this system is shown in Fig.4.9 as a reference.

During the investigation of the corrosion resistance system for the main cables of the Honshu-Shikoku Bridge Expressway Company, various methods were tried at the site, too. One of the methods in an earlier stage of investigation was the removal of the wrapping paste. Wrapping wires were unwrapped and the deteriorated wrapping paste was removed as far as possible and then the main cables were wrapped with wrapping wires again. This was because the deteriorated wrapping paste retained water and confined the humidity inside of the main cable. But after the decision of the introduction of the dry air injection system and the fact that the deteriorated wrapping paste has neither the corrosion protection function nor the corrosion acceleration function, became clear, this wrapping paste removal operation was stopped. Another example is the trial of the adoption of S-shaped wrapping wires for the existing cables. For this work, unwrapping/wrapping vehicles were developed. One of them is shown in Photo 4.13. With this vehicle, one of the sections between two cable bands of the Shimotsui-Seto Bridge was unwrapped and wrapped with S-shaped wrapping wires. The vehicle had a telescoping structure.

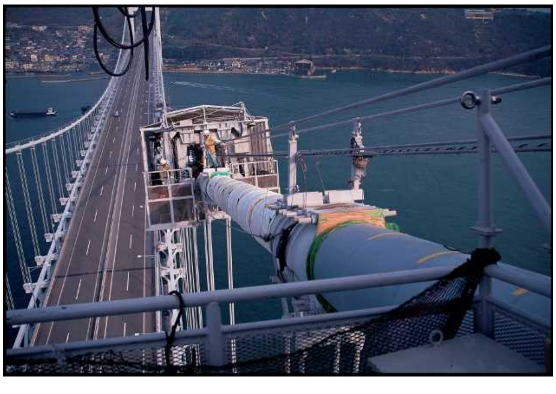


Photo 4.13 Unwrapping/wrapping vehicle

Photo 4.14 Pistons to stop vehicle

When the vehicle moves forward, the latter pistons hold the main cable firmly and the head part is pushed forward from the back side part. Pistons are shown in Photo 4.14. Then the head part is fixed on the cable by pistons and the back side part will be pulled forward by the head part. In this way the vehicle moves forward. After the unwrapping of one section and the wrapping with the S-shaped wires, this vehicle moved over the newly wrapped section. Then the adjacent S-shaped wires detached and a small gap appeared when the pistons pushed the surface of the main cable. As the void ratio of cables is about 20%, the cables will deform slightly when the cables are pressed by pistons. If small gaps appear, then the air tightness cannot be achieved. Then the adoption of the S-shaped wrapping wires for the existing cables was abandoned. Instead of S-shaped wrapping wires, the method of the flexible type paint over the main cable surfaces and the caulking of cable bands was developed and finally adopted for all of the existing suspension bridges of the Honshu-Shikoku Bridge Expressway Company.

From the above experiment, it is clear that the removal of wrapping paste is not needed and the introduction of S-shaped wrapping wires is very difficult. The same method which was adopted by the bridges of the Honshu-Shikoku Bridge Expressway Company is recommended for the Matadi Bridge, too.

Photo 4.15 Lower horizontal member

Photo 4.16 Entrance door with a lock of lower horizontal member

Photo 4.17 Inside of lower horizontal member

Photo 4.18 Dry air blower inside of tower horizontal member, Third Kurushima Kaikyo Bridge

4.7 POSITION OF DRY AIR BLOWER

In Fig.4.9, the position of the dry air blower was proposed inside of the lower horizontal member of the main tower. The lower horizontal member is shown in Photo 4.15 under the main truss girder. This place was proposed and later actually the dray air blower was installed, because this place could avoid theft as shown in Photo 4.16, the entrance was equipped with a lock. The inside of the horizontal member is shown in Photo 4.17. The prevention of theft is one of the most important problems, not only in developing countries, but also in developed countries. An example of the dry air blower inside of the tower horizontal member of the Third Kurushima Kaikyo Bridge is shown in Photo 4.18.

4.8 CABLE WIRES INSIDE OF CABLE BANDS

As discussed in Sec.2.2.2, no corrosions except for white rusts were confirmed inside of cables bands of suspension bridges of Honshu-Shikoku Bridge Expressway Company. This is also true for the Matadi Bridge as shown in Photo 4.9. But for the Matadi Bridge, a very strange phenomenon was observed as shown in Photos 4.5 to 4.8. The same phenomenon is shown again in Photo 4.19 and 4.20. The inspector of OEBK had already

Photo 4.19 Technician opened swollen paint surface

Photo 4.20 Rusty water marks.

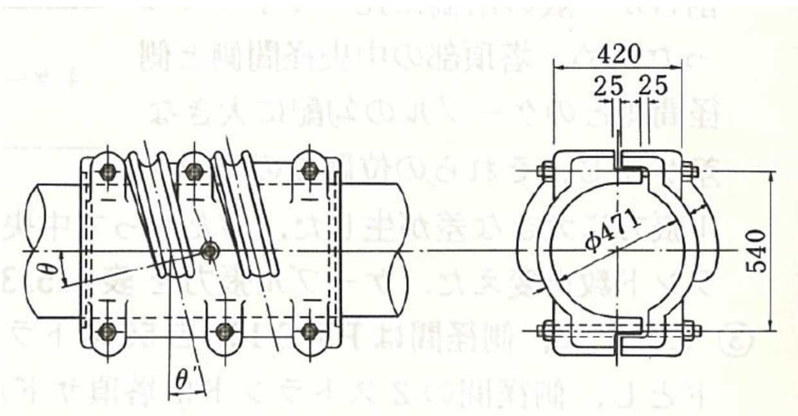


Fig.4.10 Cable band of Matadi Bridge[5)]

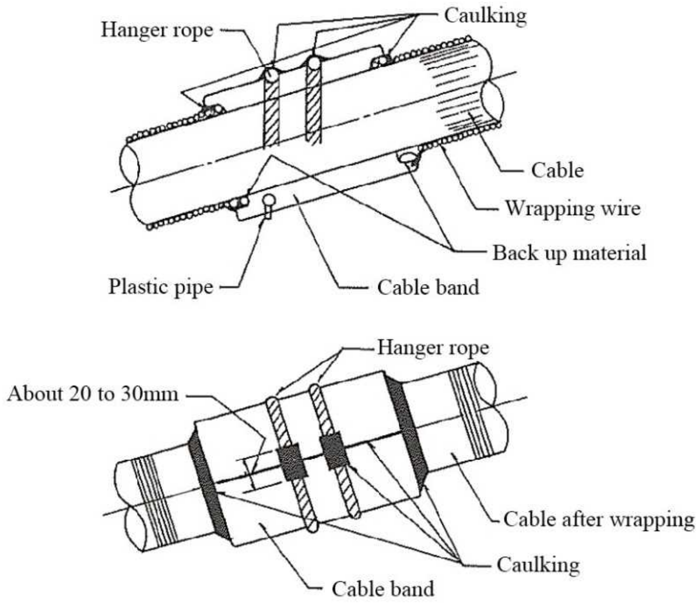


Fig.4.11 Cable band of Innoshima Bridge[6)]

Photo 4.21 Center Cable Band of Matadi Bridge and Drain Pipe

noticed the water accumulation inside of the cable. This phenomenon has never been observed for the suspension bridges of the Honshu-Shikoku Bridge Expressway Company. The reason of this phenomenon may be as follows; One example of cable bands of the Matadi Bridge is shown in Fig.4.10. This is the same type of cable bands used for the former group of suspension bridges which were constructed before 1988, namely the Innoshima Bridge, the Ohnaruto Bridge, the Ohshima Bridge, the Shimotsui-Seto Bridge, the Kita Bisan-Seto Bridge and the Minami Bisan-Seto Bridge. A cable band consists of a left half and a right half. These two halves are placed on opposite side of a main cable and they are tightened by band bolts as shown in Fig.4.10. There are an upper gap and a lower gap, through which cable wires can be seen. For the cable bands of the Honshu-Shikoku Expressway Company, the upper gaps are closed completely from the beginning by caulking material to prevent rain water. But the lower gaps are not completely closed, instead some part of the lower gaps are opened so as to drain the water inside of the main cable. This may be partly the reason why the humidity inside of the cable band is closely related to the outside air humidity as discussed in Chapter 2. The cable band of the Innoshima Bridge at the time of the wrapping wire application work is shown in Fig.4.11. In this figure, a plastic pipe is shown. This is the drain pipe of the water.

The same method of caulking must have been adopted for the cable bands of the Matadi Bridge. The center cable band is shown in Photo 4.21. In this photo, a drain pipe is seen. Therefore after the completion of the bridge, cable bands must have been well drained and the water retention phenomenon near the cable bands must not have existed. But from 2003 to 2009, the whole bridge was repainted by OEBA[7)]. During this repainting work, OEBK must have painted the cable bands completely and to some extent, closed the lower gaps, too. In Photo 4.19, the lower gap of the band is closed.

This was because the engineers of OEBK did not understand the function of lower gap opening to drain water. OEBK closely inspected the Matadi Bridge and noticed the water retention in two cable bands. This fact was reported on June 24, 2011 and the OEBK engineer was suggested to open the lower gap of the cable band immediately. When Japanese engineers revisited the Matadi Bridge in April 2012, after about one year, the lower gap was not opened yet so that it was suggested again to open the lower gaps of cable bands immediately. The result of opening of lower gaps of bands is shown in Photo 4.6 and 4.8. The fact that the lower gaps of the cable bands was left unopened for one year shows that the OEBK engineer did not understand the importance of draining water to prevent the rust. Other evidences of the lower gap closure can be seen in Photo 4.22 (the same photo as Photo 4.2.) and Photo 4.23. These photos show the lower cable band bolt caps. These caps, which were not from the band shown in Photo 4.19, were shown at the site. This shows the water leaking from the band bolts. If the lower gaps were open, this phenomenon would not occur. The water leaking from the band bolts is a good phenomenon because it can drain the water inside of cable bands. If cable band lower gaps are closed by painting, there is a possibility that the water retention can happen in all of the bands. But because of the water drain from bolts or other small gaps, only on two cable bands, the water retention was found. On the upstream side, 5th cable band from the center to Boma direction, the water retention was found,

Photo 4.22 Lower cable band bolt cap

Photo 4.23 Lower cable band bold cap

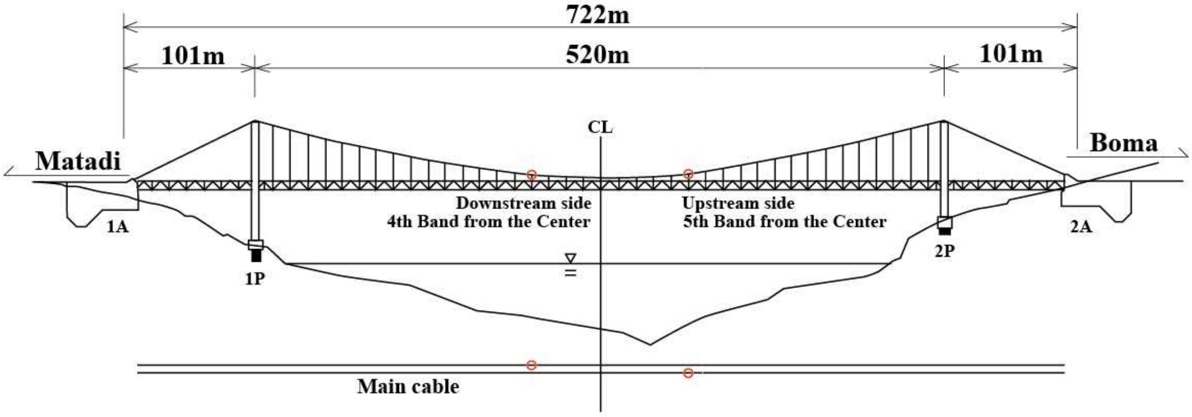


Fig.4.12 Cable bands at which water retention was found

which is shown in Photo 4.6 and 4.19. On the downstream side, 4th cable band from the center to Matadi direction, the water retention was found, which is shown in Photo 4.8. The positions of these cable bands are shown in Fig.4.12.

If a large amount of water comes into the inside of the cable from the upper part of the cable, the water flows through the cable and it will be drained at the center cable band which is equipped with drain pipes. But if only a small amount of water comes into the inside of the cable, which is generally the case, the water flows down the cable but at some point where the gravity and the viscosity of water to the cable wires balance, the water may stop. This point may correspond to the positions of 5th band of Boma side and 4th band of Matadi side in Fig.4.12. Inside of these bands, slightly more water may have remained inside of the bands compared to the upper positioned cable bands. From Fig.4.12, there are two more places opposite to the 4th and 5th bands indicated in red circles, where the water retention could have happened. But in these two places, the water retention did not occur. This might be because the water was drained better in these cable bands.

With this slightly more retained water, the rust may have proceeded more and finally retained more water. The possible process of water retention is shown in Fig.4.13. This fact strongly suggests the importance of the water drainage from the cables as soon as possible.

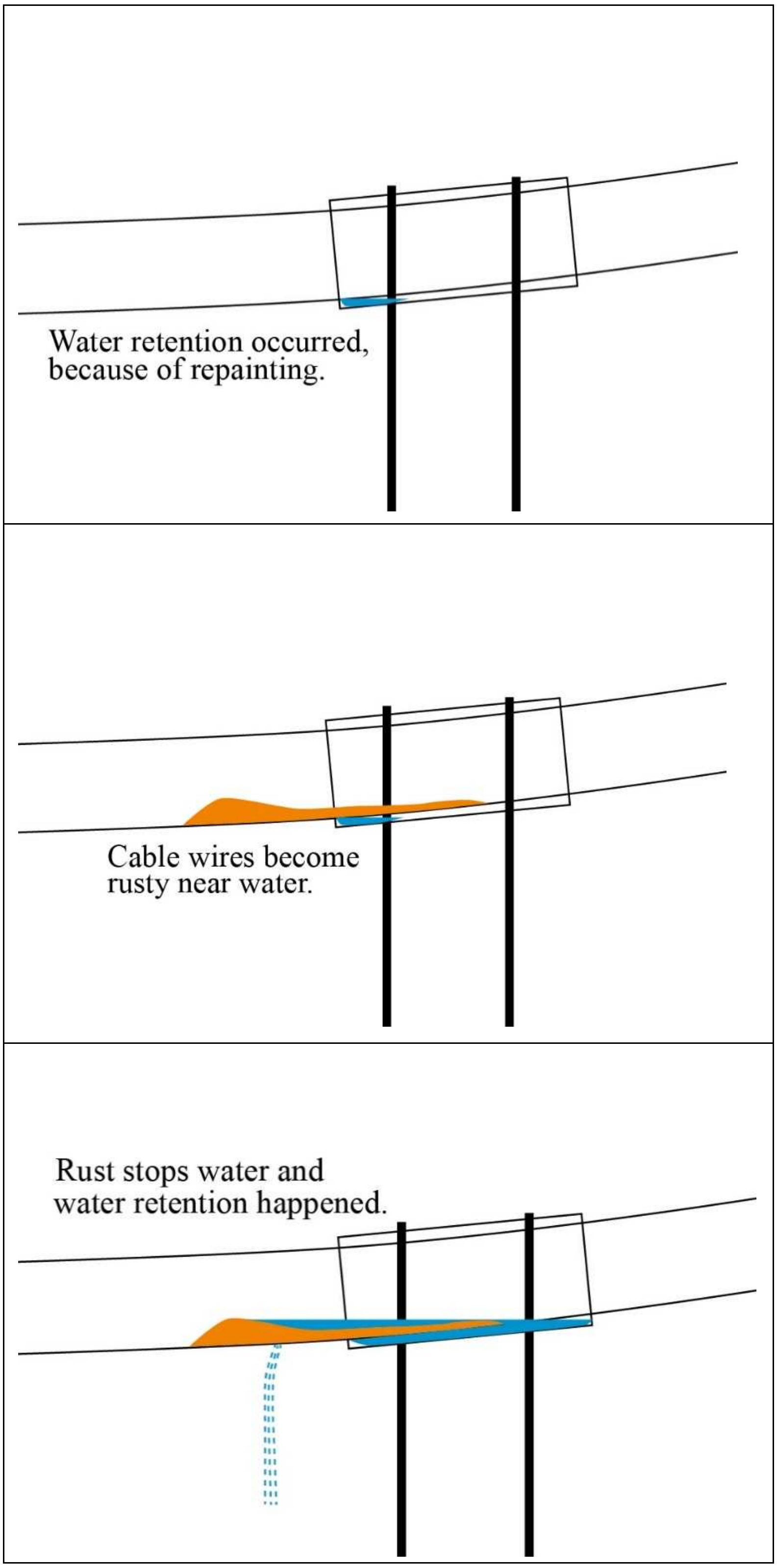


Fig.4.13 Possible process of water retention phenomenon

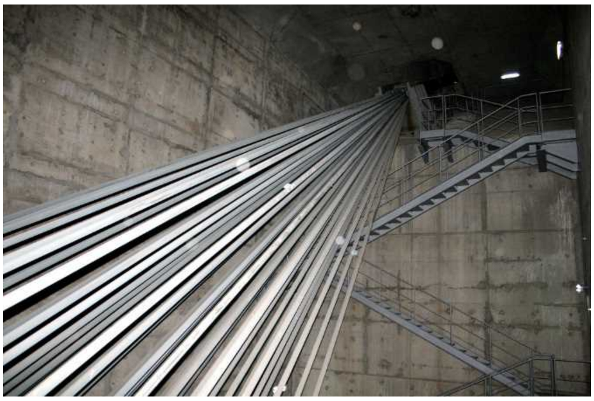


Photo 4.24 Cable strands inside of anchorage

Photo 4.25 Cable strands inside of anchorage

4.9 CABLE WIRES INSIDE OF ANCHORAGES

The cables inside of anchorages of this bridge were quite healthy except for white rust as shown in Photos 4.24 and 4.25. This is also the case for the cables inside of anchorages of the suspension bridges of the Honshu-Shikoku Bridge Expressway Company. The splay chamber of this bridge is almost closed to the outside air from the beginning so that the installation of the dehumidifier is easier. To close the chamber from the outside air, minimal work to install doors is needed. But some anchorages of the Honshu-Shikoku Bridge Expressway Company, the splay chambers were quite open to the outside air. To install the dehumidifier, instead of closing the chambers, the whole cable strands were covered with the air-tight tent structure[8)]. For the design of new suspension bridges, the splay chambers should be designed almost closed to the outside air to facilitate the installation of the dehumidifier. As discussed above, two facts are evident. The inside of cable bands cables are healthy and the cables inside of the splay chambers of the anchorages are also healthy. From these facts a new corrosion protection system of main cables can be proposed and it is discussed in Chap.6.

4.10 HANGERS OF MATADI BRIDGE

As the Matadi Bridge (completed in 1983) was constructed almost during the same period as the Innoshima Bridge (completed in 1983), therefore for the hangers, C.F.R.C rope was adopted. As discussed in Sec.2.3,

Photo 4.26 Hanger fixing structure

generally speaking C.F.R.C rope does not suffer wind-induced vibrations but the corrosion problems may happen. Compared to the hanger fixing structure of the Ohanruto Bridge, it is very easy to inspect the hanger fixing structure of the Matadi Bridge as shown in Photo 4.26. Although at present these are no corrosion problems, the regular inspection is recommended.

4.11 INSTALLATION OF DRY AIR INJECTION SYSTEM

After the main cable inspection was completed, the detailed design of the dry air injection system was performed. From November 2015 to March 2017, the installation work of the dry air injection system for the Matadi Bridge was executed and completed. At present, the dry air injection system is being operated for the Matadi Bridge. This will ensure the longer life of the Matadi Bridge.

When the Matadi Bridge was completed, a comparatively large capacity electricity supply system was introduced for the maintenance offices and the bridge lightings. After the completion of the bridge, a nearby mountain area began to be occupied by local residents and many houses were built, which are shown in Photo 4.27.

Photo 4.27 Houses constructed on mountain area after completion of bridge

According to OEBK, as the electric supply system had a large capacity, OEBK allowed to share the electricity with these houses, as in DRC, people utilize the electricity for cooking. As there are many houses at present, the amount of electricity which can be utilized by OEBK becomes too tight to operate the dry air injection system. Recently new Matadi port facilities were installed and at the same time, the electric power supply line from the hydraulic power station of the nearby Inga Dam was newly installed. OEBK changed the electric power supply line from the old line to this new line so that the dry air injection system works smoothly.

In developing countries, the stable supply of electricity is sometimes difficult. As the dry air injection system consumes comparatively large amount of electricity, the stable supply of electricity is very important. When this system is planned to be introduced in developing countries, the electricity supply needs to be investigated carefully. In the case of the Matadi Bridge, although there is a large hydraulic power station of the Inga Dam, still it was difficult to secure the electric power supply.

4.12 CONCLUSION

According to the cable opening inspection of the Matadi Bridge, the following facts become clear.

(1) Although the main cables are rusty to some extent, up to 8^{th} wire from the surface, the progress of the deterioration is slower than other suspension bridges in Japan, probably due to the less number of occurrence of dew condensation at the site. But the installation of the dry air injection system was recommended as there were two cable bands in which water was retained and consequently the cables were rusty.

(2) As the main cables are wrapped with the wrapping paste, the wrapping wires and the painting so that it is very difficult to detect the rust condition of the main cables from the outside. This was also true for the suspension bridges of the Honshu-Shikoku Bridge Expressway Company. Therefore it was reasonable that OEBK could not find out the rust inside of the main cables. OEBK regularly inspected the bridge based on the maintenance manual prepared by the contractor and checked the rust condition of the bridge.

(3) OEBK repainted the bridge by themselves during the absence of the Japanese technical cooperation. This is why the Matadi Bridge has been kept in good condition. When the Matadi Bridge was repainted, they had painted the bottom of cable bands and closed the bottom openings. In two cable bands, the wires inside had become rusty due to this closure. In the as built drawings of cable bands, only steel parts are drawn precisely. The bottom closing details by some caulking materials and the drain pipes are not shown. These details should have been shown in the as built drawings and the importance of the drain pipes should have been noted. Also in the maintenance manual, it should have been written that the water retention happened on the steel members must be eliminated as soon as possible.

(4) The engineers of OEBK noticed two places of the water retention based on the regular inspection works. They understood that the water retention was a problem but did not understand the method of solution.

(5) A simpler method to stop the progress of the cable corrosions was investigated but the wrapping paste already existed on the surfaces of the main cables. The paste confines the humidity inside of the cables. The only practical solution was the adoption of the dry air injection system. At present the dry air injection system is being operated on this bridge so that the monitoring of the humidity of the exhaust air from the exhaust bands becomes most important. If the humidity of the exhaust air is always kept below 60%, the deterioration of main cable wires will not proceed. OEBK is monitoring the humidity based on the new maintenance manual prepared in cooperation with the JICA experts. For new suspension bridges, other corrosion protection methods can be applied. This will be discussed in Chapter 6.

(6) At the center cable band, from which water was well drained, no red rust of wires was observed, only white rust was observed in the same manner as the cable bands of the former group of the Honshu-Shikoku Bridge Expressway Company. The wires inside of the center cable band of the Matadi Bridge were kept in good condition for about 29 years. The Matadi Bridge was completed in May 1983 and the wires were inspected in June 2012.

(7) Inside of anchorages, no rust of cables, except for white rust was found.

(8) When the introduction of the dry air injection system is planned in developing countries, the supply of electricity needs to be carefully investigated.

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Chapter 5 REPAVEMENT PROBLEM OF ORTHOTROPIC STEEL DECK

5.1 INTRODUCTION

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 over loading of vehicles whose axle weights sometimes exceeded 25t. Upon the advice from Japan, the General Authority for Roads, Bridges and Land Transport (GARBLT, hereinafter.) limited the axle weight of vehicles to 13t. In September 2003, after the inspection of the bridge, the bridge was handed over to Egypt. In 2011, the condition of the pavement and the steel deck was investigated by the Japan side. 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. The Japan side 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 which did not need any special machines, and actually repaved the bridge. However, this pavement was deemed viable. This pavement method could be beneficial for many developing countries, which maintain long span bridges with the orthotropic steel decks.

Photo 5.1 Suez canal bridge.

Fig.5.1 Position of Suez canal bridge.

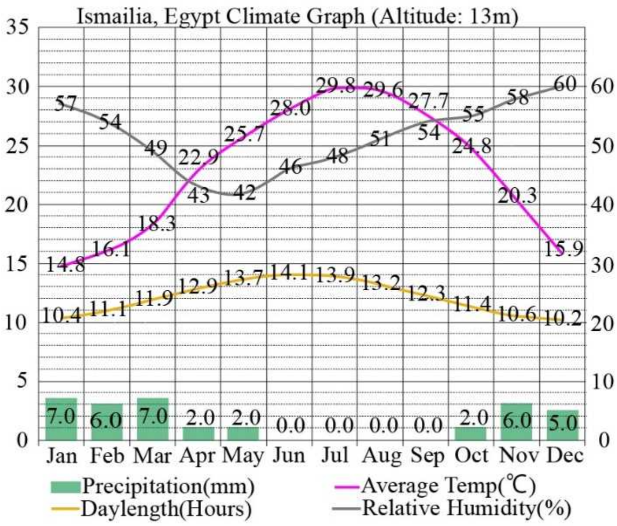


Fig.5.2 Climate chart of Ismailia[1].

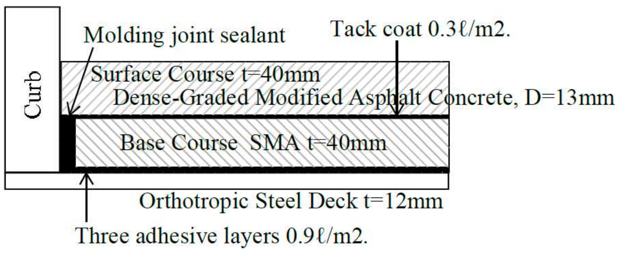


Fig.5.3 Pavement structure[2].

5.2 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 distance. 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 5.1 and the position of this bridge is shown in Fig.5.1. The bridge is between Port Said and Ismailia and in Qantara town. The bridge is situated in or near the subtropical desert biome. The climate chart is shown in Fig.5.2. The weather is dry and the total annual precipitation averages 37mm.[1] In winter from November to March, there is a little rain. However, it is common to observe fogs in the early mornings during winter.

5.3 PAVEMENT OF SUEZ CANAL BRIDGE[2]

Generally speaking, in Japan, the Gussasphalt method[3] 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.

Stone Mastic Asphalt pavement (SMA) was adopted for the base course of the pavement over the orthotropic steel deck of this bridge because SMA was utilized for the orthotropic steel deck in Japan at that time and SMA did not need any special machines. These special machines were unavailable 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.5.3. The adhesive layer consisted of three layers. Two layers of chloroprene rubber solvent adhesive, 0.3L/m^2 for the first layer and 0.4L/m^2 for the second layer were applied. As the third layer, asphalt rubber solvent adhesive of 0.2L/m^2 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. The base course (SMA) and the surface course (dense graded asphalt) 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.

5.3.1 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 whose specification is shown in Table 5.1, to achieve higher flow-resistance. The plant mix-type asphalt modifiers were used.

5.3.2 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.5.4. 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.5.4 with the median particle size. The pavement of the Suez Canal Bridge was constructed in August 2001.

Table 5.1 Standard characteristics of Type 2 polymer-modified asphalt[4).]

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

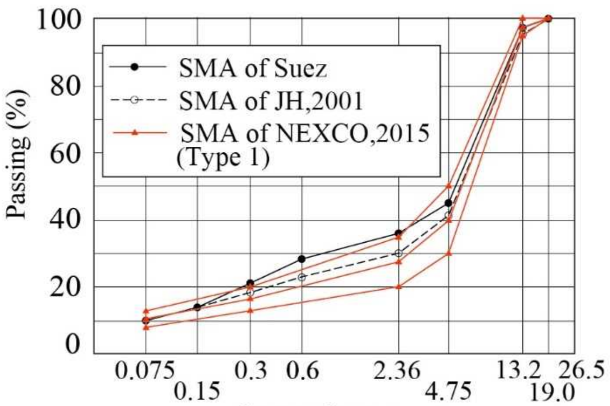


Fig.5.4 Particle size[2)].

Table 5.2 Physical property values of Suez SMA[2)].

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/cm^3)	—	2.336	2.334	2.335
Air Void (%)	≤3	2.8	2.7	2.8
Voids Filled with Asphalt (%)	75.9	84.3	83.7	84.0
stability (kg)	≥500	1,166	1,050	1,111
stability (kN)	≥4.90	11.4	10.3	10.9
flow value (1/100cm)	20-60	53	47	50

Table 5.3 Degree of compaction[2)].

Base Course		Surface Course	
Construction Date	Degree of Compaction (%)	Construction Date	Degree of Compaction (%)
Aug.2nd, 2001	96.0	Aug.8th, 2001	97.2
Aug.4th, 2001	97.0	Aug.9th, 2001	97.2
Aug.5th, 2001	98.4	Aug.11th, 2001	97.8
Ayg.6th, 2001	98.9	Ayg.12th, 2001	96.0

The physical property values of Suez SMA are shown in Table 5.2. Eight cores were sampled at the site and the degree of compaction was measured. The results as shown in Table 5.3, are satisfactory. In the paper[2)], 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.

5.4 HISTORY OF PAVEMENT AFTERWARDS

The history of the pavement of the Suez Canal Bridge is shown in Table 5.4. 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[5)]. 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 Bride 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 25t. The consultant and 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, some damage on the concrete piers constructed by the Japanese contractor were found and these damages were repaired by the Japan team. During this repair works, the pavement condition was checked in May and August 2011. The condition of the pavement was worse than in September 2003, when the defects of pavements were checked, although GARBLT continuously imposed the axle weight limitation and repaired the pavement cracks once a year.

Based on this observation, the Japanese team visited Egypt from October 2011 to March 2013 as the first site survey. During this period, the pavement 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 inspection, 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.

Table 5.4 History of pavement of Suez canal bridge and its surrounding circumstances

2016	Oct.	
	Sep.	Advice on pavement removal and repavement.
	Aug.	
	Jul.	
	Jun.	The Follow-up Cooperation Study (Counterpart Training)
	May	
	Apr.	
	Mar.	
	Feb.	Advice on pavement removal and repavement.
	Jan.	Bridge reopened only at night from 18:00 to 1:00.
2015	Dec.	
	Nov.	GARBLT decided to have the steel deck repaved by Egyptian contractors and asked JICA for assistance.
	Oct.	
	Sep.	
2013	Aug.	
	Jul.	Bridge closed due to political turnmoil in Egypt.
	Jun.	
	May	
	Apr.	
	Mar.	
	Feb.	
	Jan.	
2012	Dec.	
	Nov.	
	Oct.	As a result of this Follow-up Cooperation Study, repavement of the steel deck by Gussasphalt Method was recommended to be most suitable.
	Sep.	
	Aug.	
	Jul.	
	Jun.	
	May	
	Apr.	
	Mar.	
	Feb.	
	Jan.	The Follow-up Cooperation Study (Pavement on Steel Deck)
2011	Dec.	
	Nov.	
	Oct.	
	Sep.	
	Aug.	During FC Study, Pavement condition was surveyed.
	Jul.	
	Jun.	
	May	During FC Study, Pavement condition was surveyed.
	Apr.	
	Mar.	
	Feb.	
	Jan.	
2010	Dec.	The Follow-up Cooperation Study for the Substructures (Substructure FC Study hereinafter)
	Nov.	
	Oct.	
	Sep.	

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	Dec.	
	Nov.	Three years worth of pavement repair materials were donated.
	Oct.	
	Sep.	20th Sept. End of defects liaility period.
	Aug.	
	Jul.	
	Jun.	
	May	
	Apr.	
	Mar.	
	Feb.	
	Jan.	Ferry Service resumed.
2002	Dec.	
	Nov.	
	Oct.	Defects liability period was extended for one more year.
	Sep.	
	Aug.	13t axle weight limitation was introduced.
	Jul.	17t axle weight limitation was introduced.
	Jun.	Hair cracks on Pavement were found.
	May	Ferry Service Stopped.
	Apr.	
	Mar.	
	Feb.	
	Jan.	
2001	Dec.	
	Nov.	
	Oct.	Opening of Bridge
	Sep.	Completion of Suez Canal Bridge
	Aug.	
	Jul.	SMA Pavement on Suez Canal Bridge
	Jun.	
	May	Design and Construction Guide of SMA Pavement on Orthotropic Steel Deck, was implemented by Hanshin Expressway
	Apr.	
	Mar.	
	Feb.	
	Jan.	
1992		Hashin Expressway Company tested SMA pavement on orthotropic steel deck.[5]

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 January and September 2016, the Japanese team visited Egypt again to observe the repavement operation by GARBLT and to check the steel deck condition as the second survey.

5.5 RESULT OF FIRST SITE SURVEY OF PAVEMENT

The whole 730m bridge length was inspected precisely and recorded in 2011. It was observed that cracks were distributed over 95% of the pavement area. Furthermore cracks in approximately 70% of the bridge surface area have widths more than 10mm. The following observation was also made.

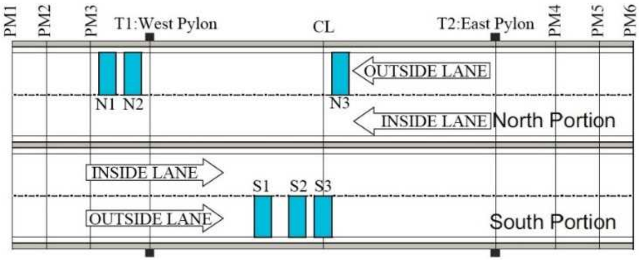
1) The outside lane of the south side lane, i.e., the east bound lane, was the most severely cracked. The heavier trucks moved from the west, Cairo side, to the east, Sinai Peninsula side. The eastbound lanes or the south side lanes had the largest total weight of the passing vehicles. This might be 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.

Severely damaged areas were counted up to 6 and inspected precisely. The areas are shown in Fig.5.5. The size of each area was 6m long and 4m wide. The inspected items were as follows: the flaking 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.5.6. One of the inspected results (N1 position in Fig.5.5) is shown in Fig.5.7. From this figure, extensive cracks can be confirmed.

The hair cracks found in June 2002 are shown in Photo 5.2. From the comparison of Fig.5.7 and Photo 5.2, the progress of the deterioration can be confirmed although the positions of Fig.5.7 and Photo 5.2 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[5)] 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 occurred mostly after three to four years.

Cracks were caused by the repetition of the traffic loads and the growth of cracks was very rapid. Compared to this observation, the deterioration of pavement of the Suez Canal Bridge was even faster. The cracks of Suez Canal Bridge appeared earlier than the potholes or the delamination. It was concluded from these facts that the cracks of the Suez Canal Bridge might have been caused by the trucks with very heavy axle weight, whose axle weight sometimes exceeded 25t. Afterwards cracks might have grown rapidly.

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.5.7, 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



(PM1~PM6 denote side span piers.)
Fig.5.5 Locations of detailed inspection.

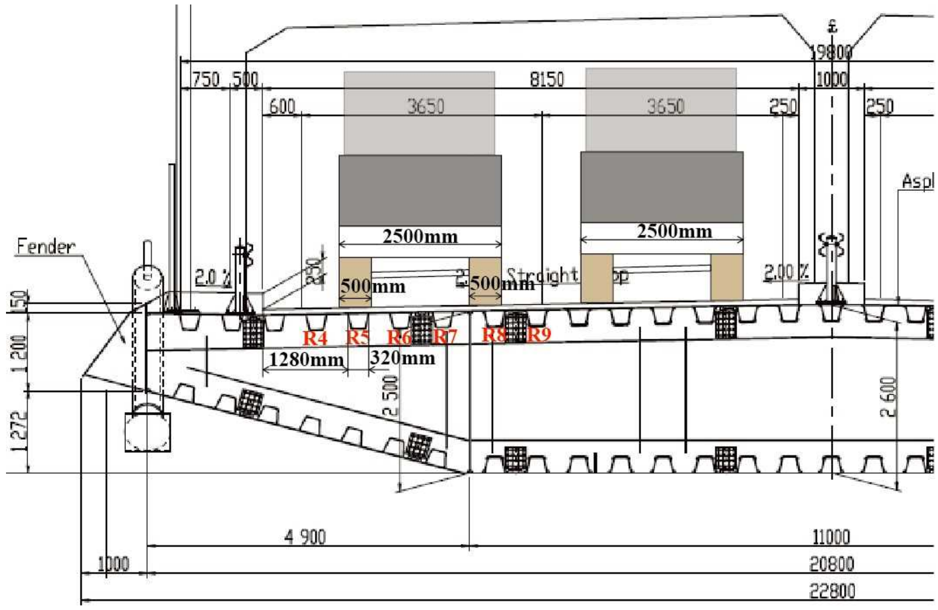


Fig.5.6 Relation between truck tire positions and U-Ribs.

cracks occurred along the bridge axis direction. In Photo 5.3, 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[5)]. It was also reported in another document[7)] 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 caused 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, not over the webs. This was quite different from the observations of bridges in Japan. The fact that cracks can occur even under compression only is proven by the same document[7)]. 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 5.3. 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

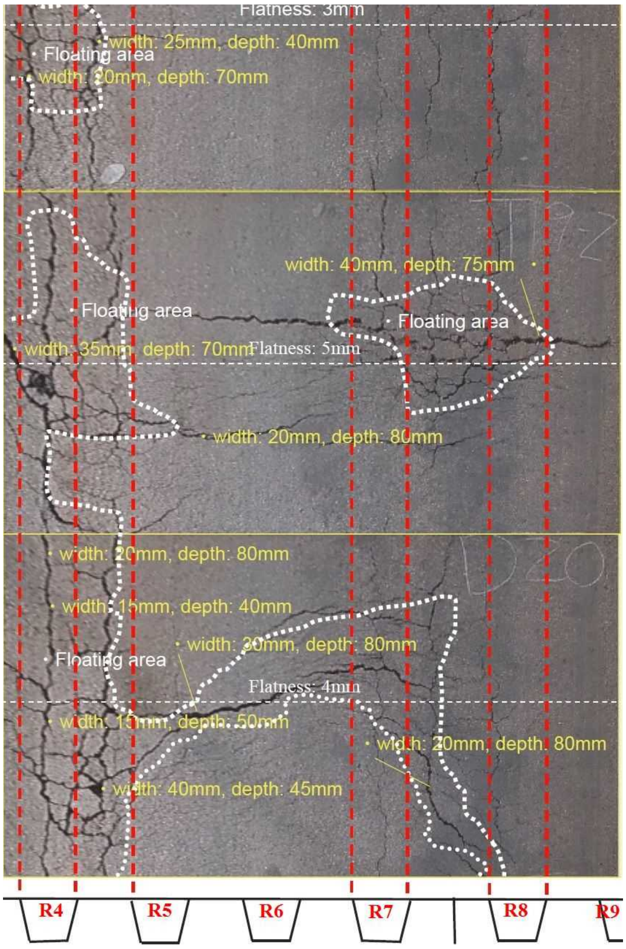


Fig.5.7 Result of detailed inspection (N1), north, outside lane.

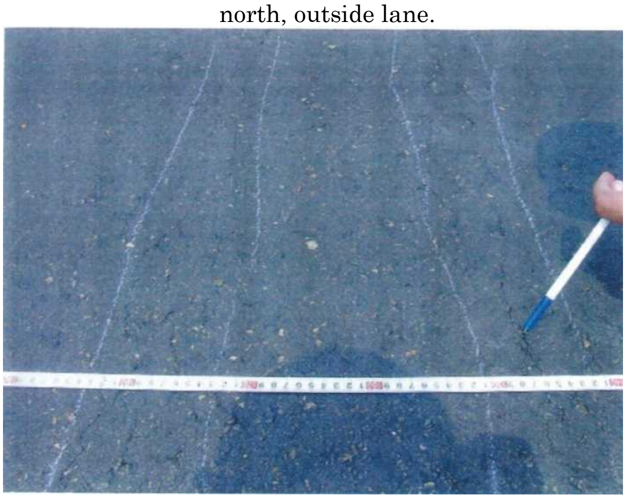


Photo 5.2 Hair cracks appeared on north, outside lane (June 21st, 2002) 6).

Photo 5.3 A crack occurred between diaphragms.

Photo 5.4 Core boring of cracked pavement.

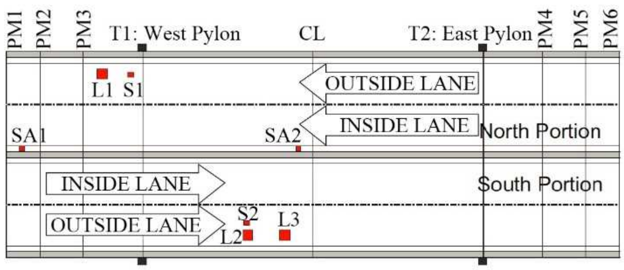


Fig.5.8 Pavement removal areas.

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.5.7.2. In places where larger cracks were found, cores with 100mm diameter were bored to examine the cracks and the

Photo 5.5 Cross-section of pavement after pavement removal (L2).

Photo 5.6 Removal of healthy pavement area (SA2).

Photo 5.7 Removed pavement (L1) and red brown substances.

Photo 5.8 L2 and S2 before pavement removal.

Photo 5.9 L2 after removal of rust.

Photo 5.10 Enlarged surface of L2 after removal of rust.

Table 5.5 Average deck plate thickness (mm).

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

adhesion to the steel deck (Photo 5.4). Six places were selected. The adhesion strengths were measured at the spot. The average measured adhesion strength was 0.113N/mm^2. This value was much smaller than 0.6N/mm^2, which was specified as the lowest allowable adhesion strength between a concrete bridge deck and a pavement.[8)] The average measured adhesion strength 0.113N/mm^2 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.5.8). They consisted of three places (L) of 1m×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 5.5. 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 5.6, 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.

5.6 RESULT OF FIRST SITE SURVEY OF STEEL DECK

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).

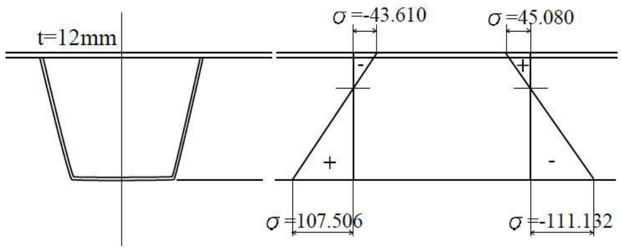
5.6.1 STEEL DECK SURFACE INSPECTION

The areas shown in Fig.5.8, 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 5.7). Only the western half of L1 was rusty and the plate thickness decreased. L2 before the removal of pavement is shown in Photo 5.8. The extensive cracking can be observed. In L2, a rust layer of about 0.5mm to 1mm stuck to the steel deck surface and was difficult to remove even with power brushes. 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 5.9. Its enlargement is shown in Photo 5.10. 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 5.5. (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 5.6. 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 σ a= 137.2MPa (=1400kg/cm²).

Table 5.6 Comparison of design stress of U-rib (σ : MPa).

Deck Plate Thickness		t=12.0mm	t=11.5mm	t=11.0mm
Bending Moment	*Mmax*	σ=107.506 (1.000)	σ=107.996 (1.005)	σ=108.388 (1.008)
	Mmin	σ= -111.132 (1.000)	σ= -111.622 (1.004)	σ= -112.112 (1.009)



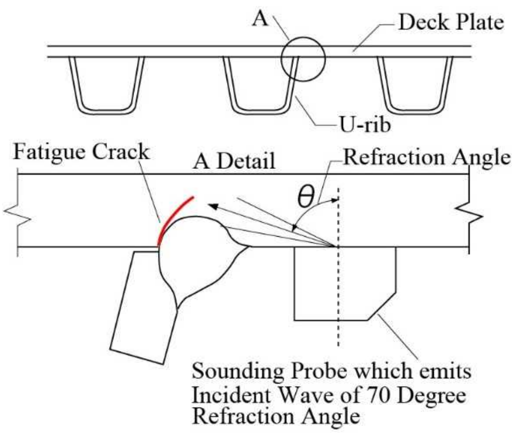


Fig.5.9 UT Test to detect fatigue cracks of orthotropic steel deck[9)].

5.6.2 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.5.9. 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[9]. 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.

5.6.3 INSTALLATION OF WATER MONITOR

During the examination of pavement cracks, seepage water above the steel deck was examined. To monitor this water, 20 water monitors shown in Fig.5.10, 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 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.

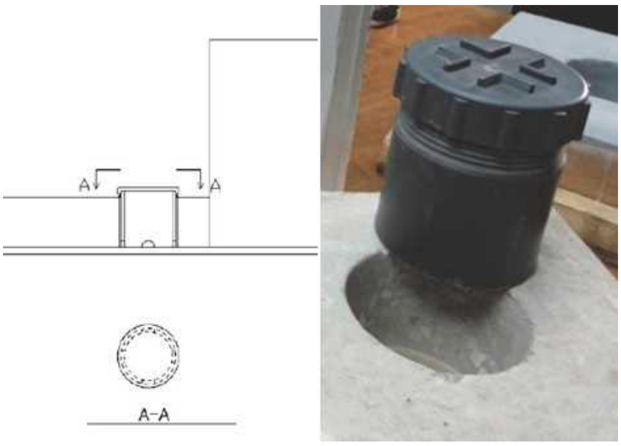


Fig.5.10 Water monitor installed at edge of pavement.

5.7 RESULT OF SECOND SITE SURVEY BEFORE REPAVEMENT

GARBLT decided to remove the pavement and repave the steel deck by its own fund in 2016. To repave the deck, a whole old pavement was needed to be removed.

In the first site survey, the pavement surfaces were investigated and the pavement of six small places was removed and the steel deck surfaces were investigated. This was because the survey was executed under the traffic. This time, the traffic was stopped because of the repair works and the pavement of a considerably larger area was removed at a time so that a larger area of steel deck conditions, the pavement conditions, seepage water conditions, etc. of the larger area could be confirmed. The reason of pavement cracks and the deterioration of steel deck surfaces were further investigated.

5.7.1 INSPECTION OF STEEL DECK SURFACE

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 5.11, 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.

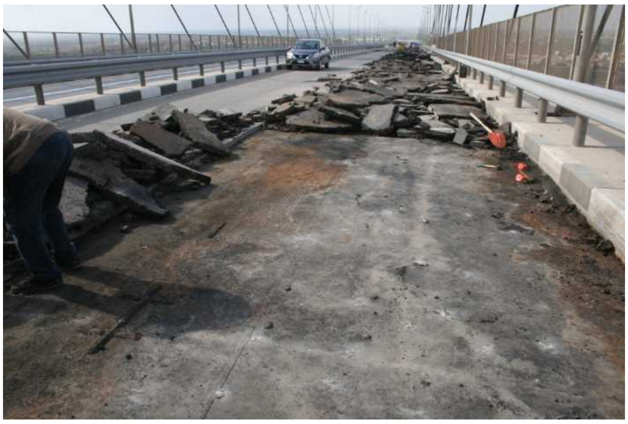


Photo 5.11 Steel deck surface after removal of pavement, west side span near west pylon.

Photo 5.12 Badly corroded area near west pylon.

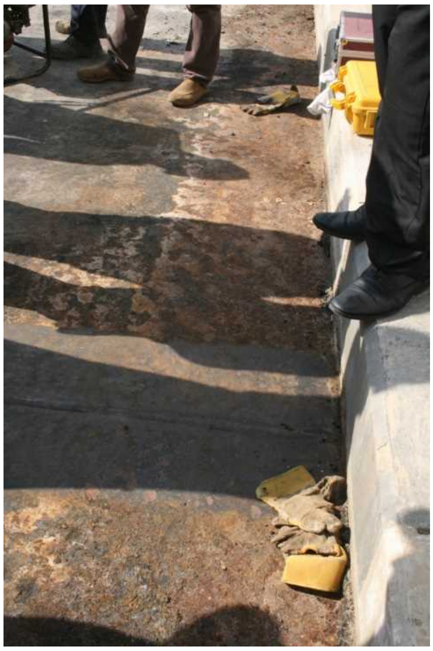


Photo 5.13 Corroded area beside curb.

The right side corrosion might have caused by the water coming from the pavement end. This is more evident in Photo 5.13. The worst corroded area was confirmed near the west pylon as shown in Photo 5.12. 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.

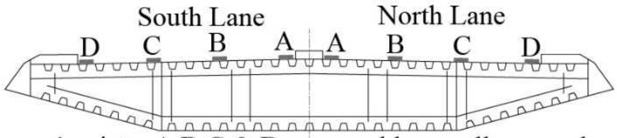
Photo 5.14 Corroded deck surface and back of pavement near center median.

Photo 5.15 Healthy deck plate, no rust is confirmed on deck and back of pavement.

Photo 5.16 Deck surface and back of pavement.

Photo 5.17 Removal of pavement.

Photo 5.18 Back of removed pavement, west side span.



4 points, A,B,C & D are roughly equally spaced.

Fig.5.11 Thickness measured points of steel deck.

Table 5.7 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

Photo 5.19 Heavily corroded deck surface, north lane, D point, 58m from west end of bridge.

Photo 5.21 Thickness measurement. Thickness was 11.21mm.

Photo 5.20 Surface where rust was removed; small part was ground flat to facilitate measurement.

Photo 5.13 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.5.3. The original molding joint sealant might have not worked well. Photo 5.14 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 5.15, 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 5.6. In Photo 5.16, 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 5. 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[10)]. 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 5.18, 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.5.11 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 5.19). The thickness was 11.21mm. The measurement of this corroded section is shown in Photos 5.20 and 5.21. 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. The result of thickness measurement is summarized in Table 5.7. 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 thick-ness of 11.00mm was found at B point and 150.5m from the west end. As shown in this table, the average thickness is 11.75mm. This value may be still admissible from the calculation of Sec.5.6.1. 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.

Photo 5.22 Largest repaired crack on south lane near center of bridge.

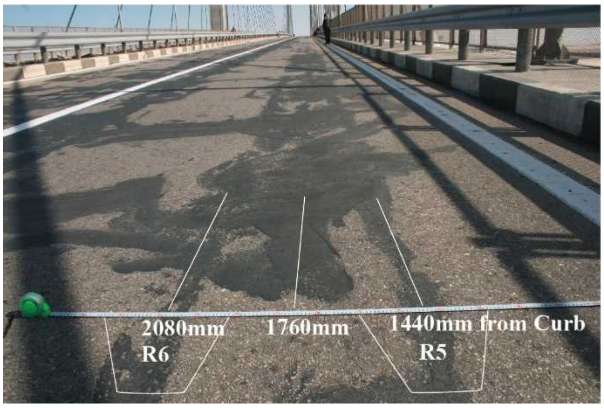


Photo 5.23 Lines of repaired crack of south, outside lane on west side span (looking east direction).

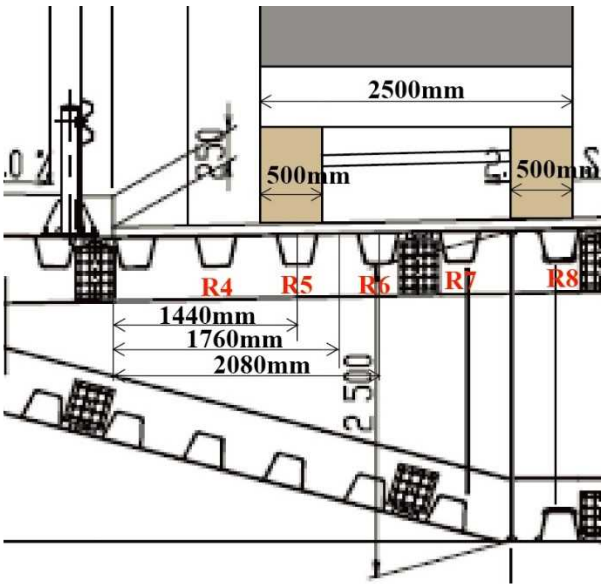


Fig.5.12 Position of R5 & R6 U-rib center (1440mm & 2080mm from curb, respectively)

5.7.2 PAVEMENT CRACKS

The repaired cracks of pavement were observed before and during the pavement removal work. The cracks were repaired 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 5.22. Other repaired cracks are shown in Photo 5.23. 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.5.12). 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 Sec.5.5.

As seen in Sec.5.6, under the cracks, seepage water and the rust were confirmed. Under the cracks shown in Photo 5.23, 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.5.7, the steel deck must have been as rusty as the steel deck plate shown in Photo 5.12. Although Fig.5.7 and Photo 5.12 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 5.13 and 5.14. In the area where no seepage water came from the pavement edges, the steel deck remained healthy as shown in Photo 5.15. 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 (Photos 5.11 and 5.23). These might be the reasons why some areas were rustier while other areas remained healthier.

5.7.3 BEARING OF WATER

As mentioned in Sec.5.6.3, 20 water monitors were installed. Their positions are shown in Fig.5.13. The observed results at noon time on January 31st, 2016 are shown in Table 5.8. 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 5.24). 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 5.25). For P12, P16, P18, and P20, higher water level was observed (Photo 5.26). In this area, all of the bottom of the pavement must have been under the water. The removed P13 monitor was examined. In Photo 5.27, 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 5.28, the rubber plate was removed and the monitor structure could be understood easily. In Photo 5.29, 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.5.2) and the 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 5.30, 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.

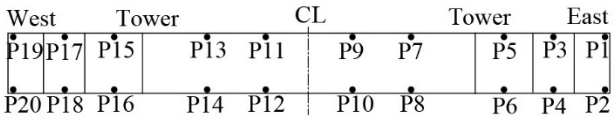


Fig.5.13 Position of water monitors.

Table 5.8 Observed results of water monitors.

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.

In summer when there are no rainfalls nor foggy days, still some water under the pavement was confirmed. 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, probably because of 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.

Photo 5.24 P10 monitor, filled with wet sand.

Photo 5.25 P11 monitor, some water and rusty deck plate.

Photo 5.26 P16 monitor, high level of water.

Photo 5.27 Removed and reversed P13 monitor.

Photo 5.28 P13 monitor, bottom rubber is removed.

Photo 5.29 Rusty deck plate below P13 monitor

Photo 5.30 Foggy morning in winter, viaduct connecting to Suez canal bridge in background.

5.7.4 CAUSE OF CRACKS AND REQUIRED CHARACTERISTICS

Compared to the Gussasphalt method, SMA pavement has less flexibility, less water tightness and weaker adhesion to the steel deck[2)]. Various measures to improve these weaker points were adopted and SMA pavement was executed for the Suez Canal Bridge[2)]. But soon after the opening of the bridge, hair cracks began to appear partly because of the overloading trucks whose axle weight exceeded 25t. As explained in Sec.5.5, the cracks developed mainly over the places between the webs of U-ribs, not directly over the webs of U-ribs. The places between webs of U-ribs deform more than those over webs of U-ribs. Therefore the loss of friction between the pavement bottom and the steel deck, and the consequent tension inside of the pavement might have resulted in the formation of cracks. This might be due to the weaker adhesion and the less flexibility of SMA pavement to the steel deck. Afterwards, the dew condensation happened inside of the cracks, especially in winter, and accumulated inside of the cracks. This water might have deteriorated further the adhesion between the steel decks and the pavement, consequently the cracks of pavement were formed all over the surface area. This water also rusted considerably the steel deck surfaces. To prevent the occurrence of these cracks, the larger adhesion of pavement to the steel deck may be the most important. If the pavement adheres to the steel deck firmly, it may be difficult to develop pavement cracks. The pavement large adhesion characteristic may be one of the most important required characteristics of the orthotropic steel deck pavement. The local method adopted by GARBLT seems to have a large adhesion of the pavement to the steel deck.

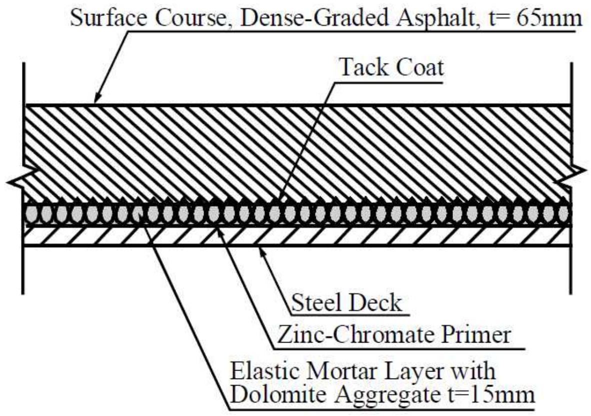


Fig.5.14 Proposed pavement method on steel deck by GARBLT.

Photo 5.31 Mixture of tar epoxy and polyurethane.

Photo 5.32 Coarse aggregates fixed by elastic mortar, resin bonded surfacing.

Photo 5.33 Galaa bridge and resin bonded surfacing.

Photo 5.34 Deteriorated resin-bonded surfacing in Cairo City.

5.8 REPAVEMENT BY LOCAL METHOD

The Gussasphalt method was first proposed, 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. Then the pavement method proposed by GARBLT, which is shown in Fig.5.14, was adopted. There is one example of application of this method. The Maasara Bridge adopted this method. On the surface of steel deck, Zinc-Chromate Primer was applied to prevent corrosions.

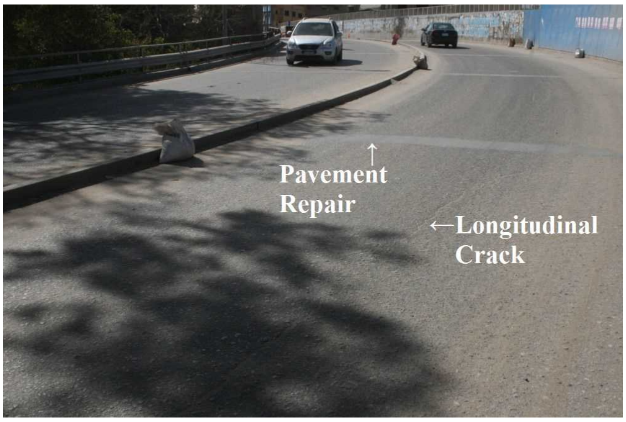


Photo 5.35 Pavement of Maasara bridge, asphalt pavement over resin bonded surfacing.


Photo 5.36 Orthotropic steel deck of Maasara bridge.


Photo 5.37 Condition of pavement and steel deck.

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 lime-stone. Although dolomite was specified, any kind of aggregates could be substituted. The elastic mortar, which was actually used for the Maasara Bridge in Cairo City and saved separately at that time, is shown in Photo 5.31. An example to show the combination of aggregates and elastic mortar over a thin steel plate, is shown in Photo 5.32. 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 5.33. Another example is shown in Photo 5.34. 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 5.35. 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)×(8 to 10m) panels as shown in Photo 5.36. These panels were connected at the site. In Photo 5.35, 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 5.35, 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 5.37. 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.5.14. 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.

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.5.14 was applied. The details of this pavement are shown in Table 5.9. 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 5.10 and 5.11, 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 5.38. The Zinc-Chromate Primer coated deck plate is shown in Photo 5.39. The finished pavement of the North Lane is shown in Photo 5.40. 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 have been proven. After about one year, no cracks have been observed. This is a better situation than that of the original pavement. After about nine months, hair cracks were observed in the original pavement.

Table 5.9 Pavement layers.

Step	Layers
5	65mm thick dense graded asphalt mixture, *D*=12.5mm, 6% bitumen content (Result at site was 5.80% to 6.20%) and 1.7% Air Voids.
4	Tack coat, 0.3L/m^2
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/m^2
1	Zinc-Chromate Primer after sand blasting.

Photo 5.38 Steel deck surface after sand-blasting.

Table 5.10 Extraction test ASTM D2172.

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	
Sieve size (inch)	Sieve size (mm)	Cumulative Retained (gm)	Retained (%)	Passing (%)	Job Mix. Tolerance	
					L.L (%)	U.L (%)
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

Photo 5.39 Zinc-chromate primer coated steel deck.

Table 5.11 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 (cm^3)	498.7	494.4
Density (gm/cm^3)	2.395	2.419
Average Density (gm/cm^3)	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	

Photo 5.40 Newly finished pavement of Suez canal bridge.

5.9 CONSIDERATION OF REPAVEMENT METHOD

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 5.34, 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 epoxy. As one good example will not assure future effectiveness, more investigations and trial tests may be needed.

5.10 MISCELLANEOUS MATTERS

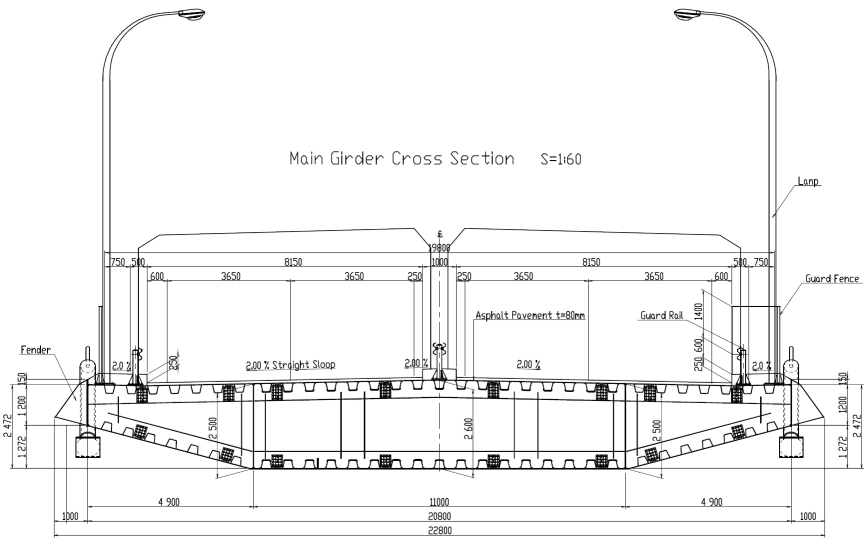


Fig.5.15 Cross section of Suez Canal Bridge

The cross section of the box girder of the Suez Canal Bridge is shown in Fig.5.15. Every 2m, there is a lateral rib of the box girder, which is not difficult to walk through as there is a large opening. Every 6m, there is

Photo 5.41 Center manhole on full-web

Photo 5.42 Manhole on stringer

A full-web diaphragm. On these diaphragms, there is only a small manhole of 750×500mm as shown in Photo 5.41 and it is very difficult to pass through these manholes.

The box girder consists of 3 cells divided by longitudinal full-web stringers. On the stringers, only small manholes of 600×400mm exist as shown in Photo 5.42. It is very difficult and laborious to pass through these manholes. As these manholes are always utilized for the inspection works, these manholes should be designed as large as possible to facilitate inspectors to pass through. As a better example, the manhole of 1100×900mm on the diaphragm of the Ikuchi Bridge is shown in Photo 5.43. This manhole is quite easy to pass through. The girder height of the Ikuchi Bridge is 2.7m at the center compared to 2.6m of the Suez Canal Bridge. The Suez Canal Bridge could have adopted larger manholes. In Photo 5.44, another example is shown. The box girder of the Third Kurushima Kaikyo Bridge. This bridge adopted the truss structure diaphragms, which is easy to walk through. It is important for bridge designers to design the inside of box girders so that the maintenance works can be done easily, otherwise inspectors do not want to go inside.

Photo 5.43 Manhole of Ikuchi Bridge

Photo 5.44 Truss structure diaphragms of Third Kurushima Kaikyo Bridge

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 5.45), 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 5.46, were found. The condition of the girder was quite good after about 15 years because of the dry weather as seen in Fig.5.2. 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 5.47 and 5.48). 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 is not a structural member, it is better to repair and repaint the coating as soon as possible.

Photo 5.45 Bottom flange of box girder, although covered with fine sand, condition was good.

Photo 5.47 Severe corrosion found on welding bead of wind fairing.

Photo 5.46 Spot rust on bottom flange, this kind of rust was sometimes found.

Photo 5.48 Corrosion of **Photo 5.47** seen from beneath.

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 5.49. Later the cuts were smoothened to avoid stress concentration and to erase the possible cause of the fatigue cracks. The smoothened result is shown in Photo 5.50.

Photo 5.49 Cutting hanging hook and steel deck by pavement cutting machine.

Photo 5.50 Smoothened surface of cuts.

5.11 CONCLUSION

According to the inspection of the orthotropic steel deck pavement of the Suez Canal Bridge, the following facts become clear.

(1) 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 investigation. 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.

(2) Although the Gussasphalt method for the repavement of the bridge was recommended by the Japanese team,

GARBLT refused to adopt this method because the special machines needed for the Gussasphalt method were unavailable in Egypt. The old SMA pavement of the Suez Canal Bridge was removed and the bridge was repaved by the own method of GARBLT, which did not need any special machines. The own method had the successful record of more than 20 years for the Maasara Bridge in Cairo city. This pavement method could be successful but needs to be monitored for the next few years to confirm its effectiveness. If this method is durable and effective, this method is recommended as the repavement method of the orthtropic steel decks in developing countries. Or from the beginning this method can be adopted for the orthtropic steel deck bridges in developing countries as no special machines are needed.

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

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Chapter 6. FEEDBACK FOR DESIGN OF LONG SPAN BRIDGES

Based on the results of study in Chapter 3, 4 and 5, the following design methods are proposed. They are the design ideas of inspection vehicles, the new cable corrosion protection system and the new orthotropic steel deck pavement method.

6.1 DESIGN OF INSPECTION VEHICLES

In Chapter 3, the importance of the adoption of manually driven inspection vehicles was found. In this section, from the repair works of the manually driven inspection vehicles of the Matadi Bridge, the importance of the redundancy of driving gears of inspection vehicles is stressed.

When the OEBK engineers moved the under girder inspection vehicles, they almost always moved two under girder inspection vehicles together as shown in Photo 3.2. A part of Photo 3.2 is enlarged in Photo 6.1.

Photo 6.1 Two vehicles tied together

In the photo, two vehicles were tied together by ropes at two points. But in the real situation, two vehicles were tied at three points, at the center, the upstream side and the down stream side. The downstream side is not shown in the photo. The reason why these two inspection vehicles were tied together might be as follows; there were four driving wheels for each under girder inspection vehicle. In total eight driving wheels for two inspection vehicles. Out of eight driving wheels, at least two driving wheels had been already broken at that time. To increase the number of driving wheels, the OEBK inspectors combined the two inspection vehicles together and moved them. This suggests the necessity of redundancy of the number of driving wheels.

The manual driving gears can be used for a long time compared to the electrified driving gears because the maintenance is easier as there are no electrified parts and the possibility of theft of parts is low. This may be the reason why the inspection vehicles of the Matadi Bridge can be utilized for nearly 30 years. But still some manual driving wheels had been broken and the OEBK inspectors tried hard to cope with this situation.

The original configuration of the driving gears is shown in Fig.6.1. All of the four driving wheels were placed outside of the rails. Therefore if the rails were inclined, the detachment of driving wheels happened. As explained in Sec.3.3.1, the fact that the outside driving wheel detached from the rail surface suggested that the inside supporting wheels were pressed more against the rail surface. From this fact, the both sided driving gear was proposed and adopted for the repair of the Matadi Bridge as shown in Fig.6.2.

The configuration which was actually adopted for the repair of the Matadi Bridge is recommended for the future design of manually driven inspection vehicles in developing countries, not only because this configuration can move easily over the inclined rails but also because this configuration has a higher redundancy of driving wheels. The number of driving wheels of the configuration shown in Fig.6.2 is eight compared to four shown in Fig.6.1. The configuration of Fig.6.2 has a double redundancy of the configuration of Fig.6.1. As one inspection vehicle has eight driving wheels, even if one or two driving wheels are broken during the course of long term maintenance, still it is possible to utilize one inspection vehicle alone, this allows the inspection team to inspect bridge girder sections more effectively.

On each side span of the Matadi Bridge, one new under girder inspection vehicle with the driving gear configuration of Fig.6.2 was installed. As one inspection vehicle has eight driving wheels, even if one or two driving gears will be broken, still it will be possible to drive the vehicle alone. The redundancy of the driving gears of one new inspection vehicle is the same as the redundancy of the two center span under girder inspection vehicles with the original driving gear configuration of Fig.6.1. As on each side span, there is only one under girder inspection vehicle, this redundancy is very important.

For the future design of manually driven inspection vehicles, this redundancy of driving wheels should be adopted.

As discussed in Sec.3.3.2, inspection vehicles with manual driving gears are difficult to be adopted for bridge girders which have more than 1% longitudinal gradient. The road suspension bridges or the road cable-stayed bridges often adopt the longitudinal gradient of 3%. In this case, the manually driven inspection vehicles cannot be adopted. The electrified inspection vehicles need to be adopted. One example of the electrified inspection vehicle is shown in Fig.6.3 and Photos 6.2, 6.3, 6.4.

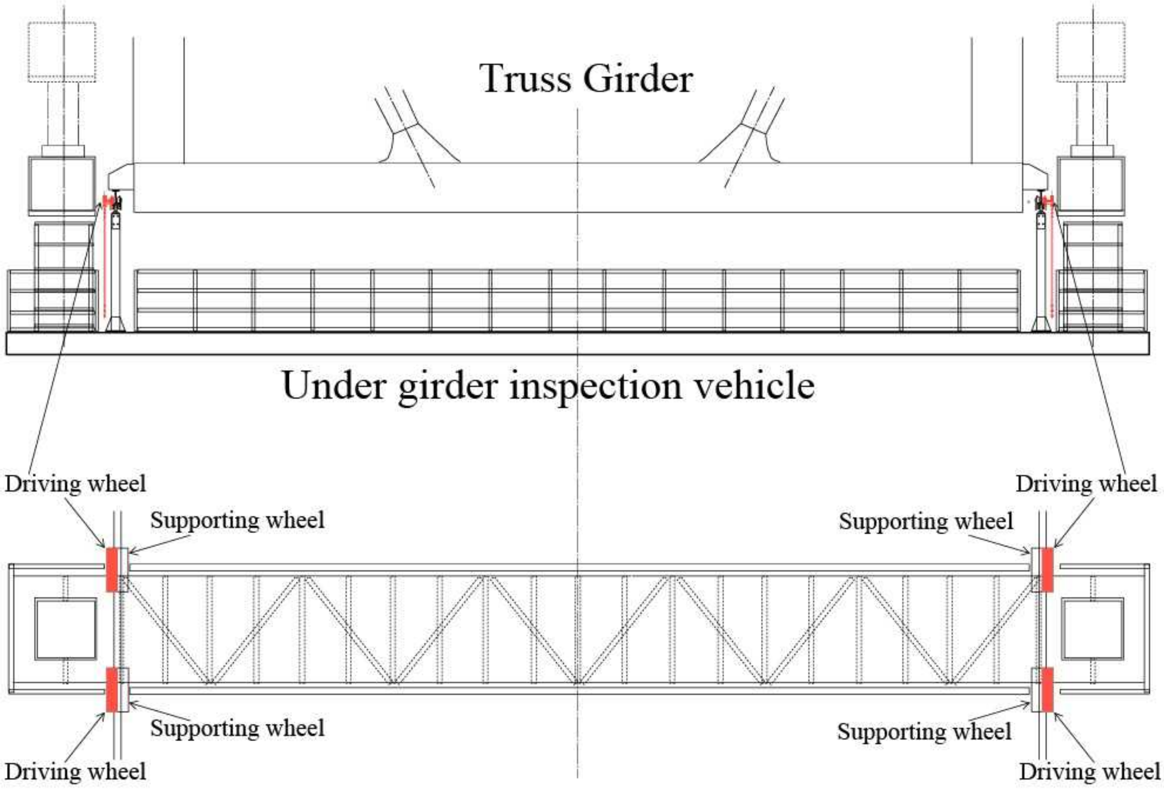


Fig.6.1 Original configuration of driving gears, Matadi Bridge

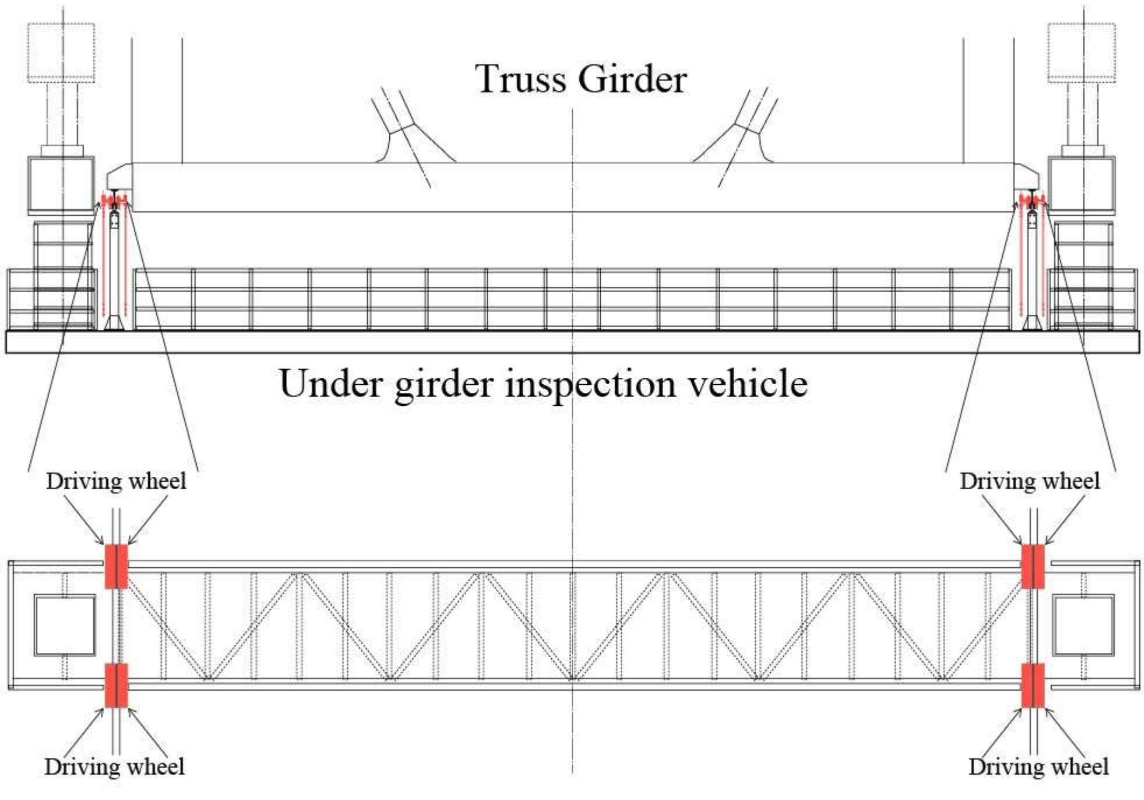


Fig.6.2 Configuration of driving gears after improvement, Matadi Bridge

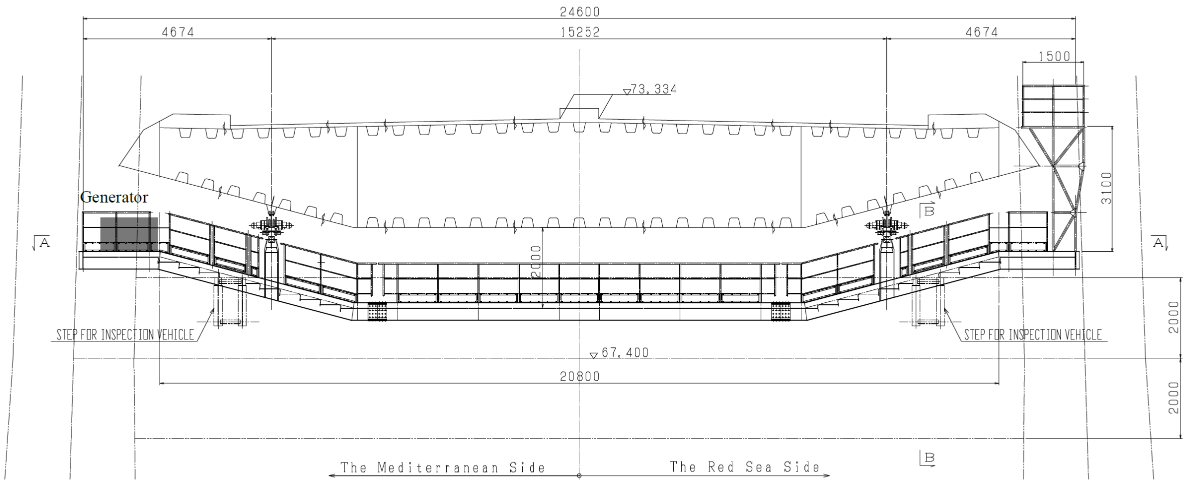


Fig.6.3 Inspection Vehicle of Suez Canal Bridge

This is the inspection vehicle of the Suez Canal Bridge. This inspection vehicle is electrified with the generator on the north side (left side in Fig.6.3.) of the vehicle as shown in Photo 6.3. On the south side of the vehicle, (right side in Fig.6.3.) instead of generator, there is a scaffolding to facilitate the inspection of the girder side. (Photo 6.4) Because of this scaffolding, the south side is easier to inspect than the north side. But from the point of view of maintenance, the same scaffolding on the north side is recommended. The generator should have been placed at the center of the vehicle, although this would push up the cost of the inspection vehicle.

As there are no trolley lines which are difficult to maintain, no problem was found for the inspection vehicle. The Suez Canal Bridge is heavily guarded by the police, always three to four policemen are on the bridge, so that the possibility of theft is very low. Also the bridge has a very high clearance, 70m, from the sea level, so that the access from the ground is very difficult, too. The police boxes are shown in Photo 6.5. Because of these reasons, the theft problem did not happen.

When electrified inspection vehicles are planned, a careful planning of theft prevention measures is very important. The recommended countermeasures are as follows;

(1) The detachable electric parts. If the electric motors, batteries, etc. are designed as detachable, then it is possible to bring back these electric parts to the safe storage after the inspection work.

(2) If a large generator, such as the one shown in Photo 6.3, is planned to be installed, the generator should be protected firmly by the outer shell to avoid the theft and the vandalism.

(3) Arrangement of security guards for all day long to prevent the theft.

Photo 6.2 Inspection vehicle of Suez Canal Bridge

Photo 6.3 Generator of north side

Photo 6.4 Inspection scaffolding of south side

Photo 6.5 Police box on bridge

6.2 CABLE CORROSION PROTECTION SYSTEM

6.2.1 PROPOSAL FOR NEW CABLE CORROSION PROTECTION SYSTEM

There are three types of the corrosion protection system for the suspension bridges of the Honshu-Shikoku Bridge Expressway Company. They are the system of former group bridges, that of the Akashi Kaikyo Bridge and that of the Kurushima Kaikyo Bridges. In all three types, it has been tried to achieve the air tightness of the cable surfaces as strong as possible. The cable bands of the former group were not air tight in the beginning but they were caulked when the dry air injection system was introduced, also the cable surfaces were painted by the soft type paint, to prevent the air leaking from the cable bands and other cable surfaces.

The dry air injection system assumes the closed air tight system. But if the results of inspection of the cable opening are checked, the following fact can be noted that “(6) Inside of cable bands, cables were healthy except for some white-rust, which were confirmed on the lower side of cables” in Sec. 2.2.2. This was also true for the center cable band of the Matadi Bridge as shown in Sec. 4.5.

This observation can be utilized for the new design of the corrosion protection system of main cables. The wrapping system of the former group is shown in Fig.4.3. The improved wrapping system without the wrapping paste is shown in Fig.6.4. The wrapping system of the Akashi Kaikyo Bridge, the rubber wrapping system, is shown in Fig.6.5. The wrapping system of the Kurushima Kaikyo Bridges, the S-shaped wrapping wire system, is shown in Fig.6.6. The S-shaped wrapping wire system can achieve the higher air tightness than the round shape wrapping wire system because of the S-shaped wires, therefore the rubber wrapping was omitted, hence the S-shaped wire wrapping system is easier to maintain and superior to the rubber wrapping system of the Akashi Kaikyo Bridge.

The steel cover cable corrosion protection system which is proposed in this study, is shown in Fig.6.7. This system is proposed based on the observations of cable bands of the former group bridges and the Matadi Bridge. Inside of cable bands, no rust of wires was found. This may be because inside of the cable bands, even if rain water comes in from the upper side, it will be drained soon from the bottom side. In this system, main cables are covered by the steel plate covers instead of wrapping wires.

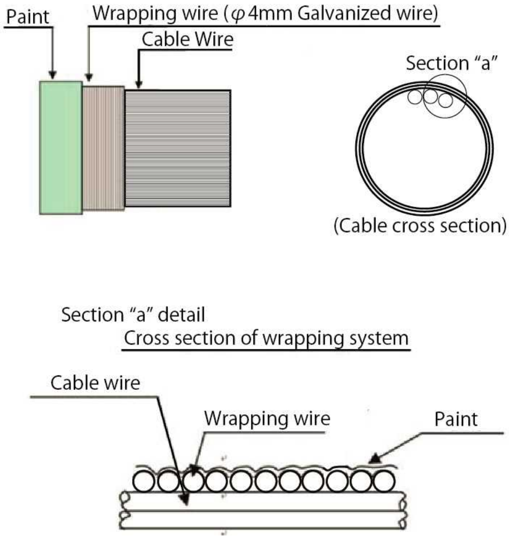


Fig.6.4 Wrapping system without paste

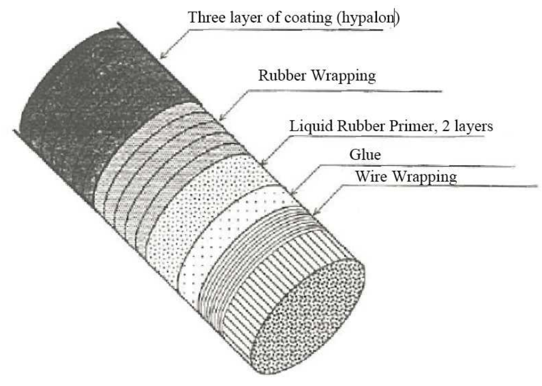


Fig.6.5 Wrapping system of Akashi Kaikyo Bridge, Rubber wrapping[1]

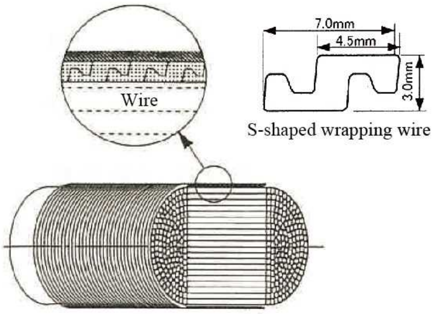


Fig.6.6 S-shaped wrapping wire system for Kurushima Kaikyo Bridges[2]

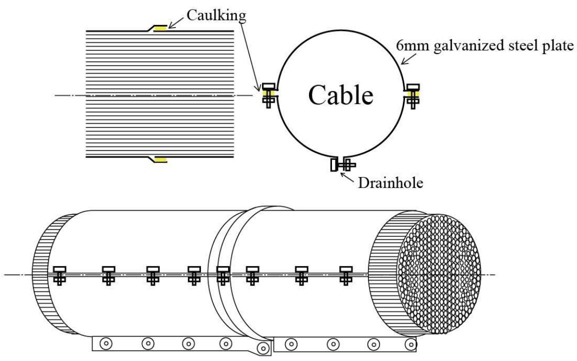


Fig.6.7 Steel cover corrosion protection system

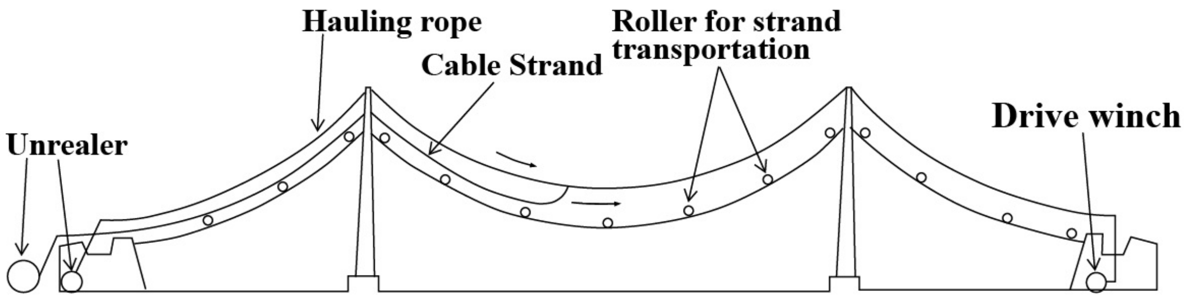


Fig.6.8 Construction of main cable[3]

When a suspension bridge is constructed, first, towers and cable anchorages are constructed. Then main cables are constructed utilizing the hauling system spanned above the towers and anchorages as shown in Fig.6.8. After the erection of all of the strands, main cables are squeezed to have a round shape, one example of which is shown in Photo 6.6.

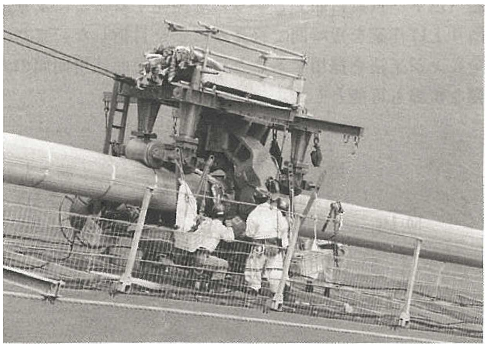


Photo 6.6 Cable squeezing of Second Kurushima Kaikyo Bridge[4)]

Then cable bands are placed along the main cables and band bolts are tightened to receive hangers on the cable bands. Then the stiffening girders are constructed and fixed. After the erection of the stiffening girder, the main cables are wrapped by wrapping wires to protect the cable surfaces and to shut out rain water.

For the former group suspension bridges and the Akashi Kaikyo Bridge, the main cables were wrapped by round shaped wrapping wires. Instead of these wrapping wires, the system of steel cable covers is proposed, which is shown in Fig.6.7. Main cables along the whole length will be covered by these steel cable covers except for the cable band sections. The application of this system is shown in Fig.6.9. One steel cover consists of three pieces and upper two bolt connections will be caulked to prevent rain water but the lower connection will be left open so that even if rain water comes into the inside of the cable, it will be drained from the bottom connection easily.

A steel cover which consists of three pieces is proposed. This is because over these covers, inspectors need to walk as shown in Photo 6.7 and the flat surface is needed. As the cables inside of cable bands of the Matadi Bridge is healthy for about 30 years, this steel cover system is also expected to protect cables for more than 30 years without any electric systems, especially under the similar tropical climate in which the Matadi Bridge is situated. The merit of this system is that it is not necessary to monitor the humidity of the exhaust air from the inside of the cables. The electrified machines of the dry air injection system need to be repaired or replaced after some years when the machines are deteriorated. But if some water retentions are found, the cause of the water retentions must be removed as soon as possible.

For this steel cover corrosion protection system, if some corrosions are found such as the corrosion shown in Photo 4.6, it is easy to introduce the dry air injection system in the same manner as adopted for the existing cables of the former group. First the steel cable covers should be caulked to achieve the air tightness. Then the dry air injection system can be installed by replacing several steel covers by the dry air injection covers and the exhaust covers. One example of the exhaust cover of the Kurushima Kaikyo Bridge is shown in Photo 6.8.

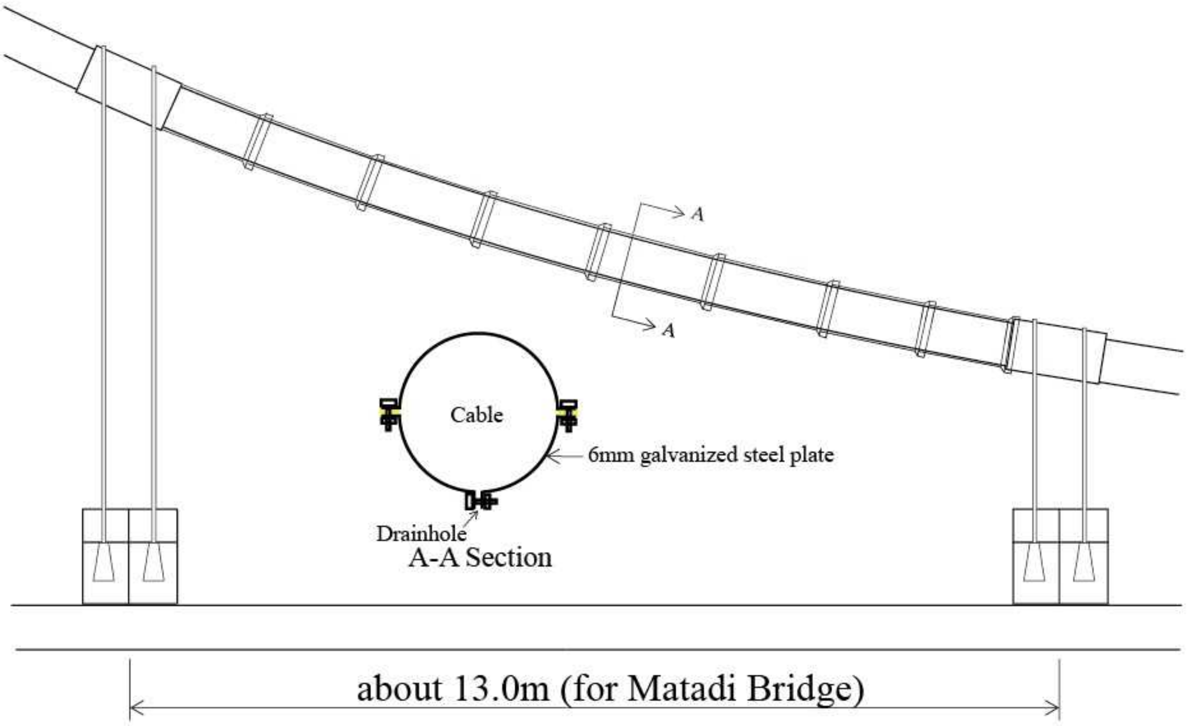


Fig.6.9 New corrosion protection system

Photo 6.7 Inspection of Main Cable,
Matadi Bridge

Photo 6.8 Center Exaust Cover,
Third Kurushima Kaikyo Bridge

This system is applicable for the new bridges only. If the wrapping paste exists, the wrapping paste will confine the humidity inside the cable and this system will not work. The present dry air injection system is first developed to prevent the further deterioration of the wires inside of the existing cables wrapped by the wrapping paste.

6.2.2 COMPARISON OF CABLE CORROSION PROTECTION SYSTEMS

For the design of the new suspension bridges, there are three alternatives for the corrosion protection system of main cables. Three alternatives are, Alternative (1) Improved conventional system without the wrapping paste shown in Fig.6.4, Alternative (2) S-shaped wrapping wire system in Fig.6.6 and Alternative (3) Steel cover corrosion protection system shown in Fig.6.7. These three alternatives are compared in Table 6.1. As the S-shaped wrapping wire system is better than the rubber wrapping system, the rubber wrapping system is not included in the comparison.

Alternative (1) is almost the same as the system which is adopted for the present former group suspension bridges except for the wrapping paste. The air tightness is secured by the soft type paint over the wrapping wires and the caulking of cable bands. As this system is presently adopted, the effectiveness of corrosion protection is already proved. For the repair of the existing suspension bridge cables, which adopted the wrapping system shown in Fig.4.3 and utilizing the wrapping paste, this is virtually the only method applicable.

Alternative (2) has been adopted by the Kurushima Kaikyo Bridges and its effectiveness is already proved. This alternative is more expensive than Alternative (1), as the S-Shaped wires are more expensive than the conventional round shaped wires although the air tightness is higher.

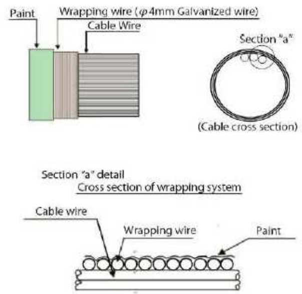
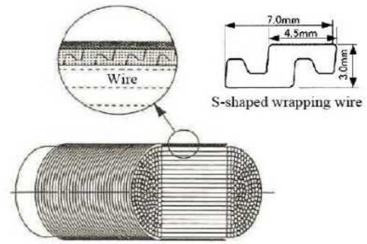
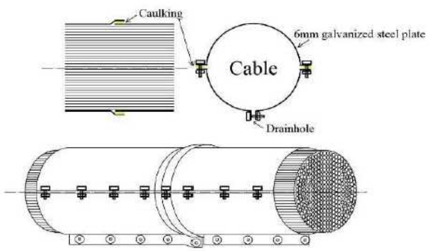
Alternative (3) can be named as a half closed system. Although there is no real application of this method, the effectiveness of this method is already half proved by the fact that no rusts were observed for the cables inside of the well-drained cable bands, such as the cable bands of the former group suspension bridges and the Matadi Bridge. If this method is adopted for the new bridges in Japan, at least for 7 years, the cables should be kept healthy from the inspection results of the suspension bridges of the Honshu-Shkoku Bridge Expressway Company. If this method is adopted for the new bridges in the area whose climate is similar to the equatorial climate of the Matadi Bridge, the cables should be kept healthy for about 30 years,

It is quite easy to inspect the cables of this method from the bottom of the cable bands and covers. It is not necessary to open the cables to inspect the inside. If this method is adopted in developing countries, it is mandatory to teach the local maintenance engineers not to close the bottom of cable bands and cable covers. If some water retention is found, they need to immediately drain the water from the place of water retention.

The half closed system does not employ the dry air injection system hence the maintenance is easier and the construction cost is smaller. Also it is not necessary to exchange electric machines inside of the dry air injection system after some years. For Alternative (1) and (2), it is necessary to maintain the dry air injection system, to monitor the exhaust air and its humidity and to renew caulking materials often as discussed in Sec.2.2.6. Therefore for the new bridges, especially in the similar climate as the area of the Matadi Bridge, the adoption of the Alternative (3) is recommended.

For the design of new suspension bridges, the splay chambers should be designed almost closed to the outside air to facilitate the installation of the dehumidifier in the future. In the splay chambers of the anchorages, dehumidifiers may not need to be installed from the beginning. As discussed in Chapter 4, the cables inside of the anchorages were healthy. The dehumidifier can be installed after the inspection for several years and when the necessity arises.

Table 6.1 Comparison of Cable Corrosion Protection Systems

Alternative	Figure	Wrapping system	Dry air injection system	Mechanism	Cost	Applicability		Comment
						existing bridge repair	new bridges	
Alternative (1)		φ4mm wrapping wire+ soft type paint	Installed from the beginning	Air tightness is secured by the soft type paint and the caulking of cable bands. Normal round wrapping wire is utilized.	expensive	Yes	Yes	Applied examples exist. Its effectiveness is already proved. Maintenance of electrified machines is needed.
Alternative (2)		S-shaped wrapping wire+ soft type paint	Installed from the beginning	Air tightness is secured by the soft type paint, S-shaped wire and the caulking of cable bands. This system is utilized by Kurushima Kaikyo Bridges.	most expensive	No.	Yes	Applied examples exist. Its effectiveness is already proved. Maintenance of electrified machines is needed. Air tightness is higher because of S-shaped wires.
Alternative (3)		Steel cable cover	No dry air injection system.	Rain water from upper side will be drained soon from the steel cover bottom. Newly proposed.	cheaper	No.	Recommended	Maintenance is easier. In tropical area, similar to the area of Matadi Bridge, this system may work for 30 years. In Japan, at least 7 years, this system may work well from the inspection result of Innoshima Bridge.

6.3 ORTHOTROPIC STEEL DECK PAVEMENT METHOD

In Chapter 5, the orthotropic steel deck pavement adopted by GARBLT was introduced. This method can be proved to be durable in the near future. The SMA pavement adopted for the Suez Canal Bridge, unfortunately did not last for a long time. It developed extensive cracks within ten years after the completion. Then the Gussasphalt method was recommended. But to adopt the Gussasphalt method, special machines and Trinidad Tobago Lake Asphalt (TLA) were needed and the other method proposed by GARBLT was adopted.

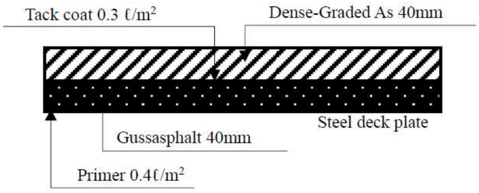
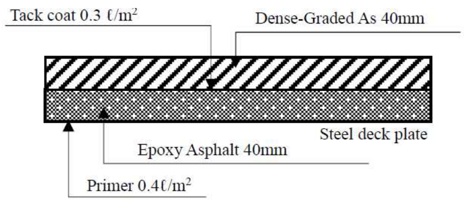
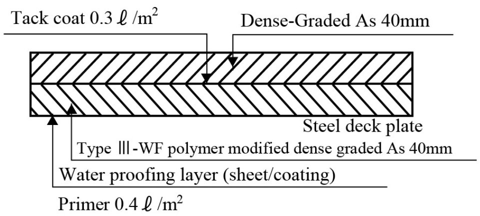
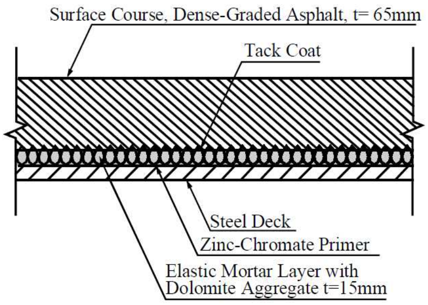
According to the paper[5)], when the orthotropic steel deck pavement of the San Mateo-Hayward Bridge in California, U.S.A., was studied, the preceding examples were investigated. One of them was the Port Mann Bridge, a steel arch bridge with an orthotropic steel deck constructed in 1964. This bridge deck was paved with asphaltic concrete over an epoxy red lead corrosion protective coat and an epoxy coal tar bond coat containing embedded grit. The Port Mann pavement is regarded as completely successful. It is also reported on Page 15 that "The Port Mann was apparently proceeding satisfactorily with a different multilayered system: a red lead epoxy polyamide paint overlaid with epoxy coal tar with grit embedded to act as a "tooth" for the overlaying asphalt concrete. " The proposed method of GARBLT is closely the same as this method. But for the study of the pavement of San Mateo-Hayward Bridge, this method was not included as the simpler system involving only one type material was more preferable. Finally epoxy asphalt was chosen. From the fact of the Port Mann Bridge, the pavement method proposed by GARBLT may be feasible not only in Egypt but also in other countries.

The epoxy asphalt pavement was developed in USA and it is utilized for many bridges in USA. But the temperature control during the construction is difficult[6)]. This method is also developed in Japan but only a few examples of application exist. The construction cost of this pavement should be cheaper than the Gussasphalt method as the special machines are not needed[7)]. As there are not so many examples of application in Japan, it would be difficult to adopt this method for overseas Japanese ODA project in developing countries by the Japanese contractors.

In "Guideline of pavement design and construction, 2006", Japan road association, the modified asphalt for orthotropic steel deck was renamed to TypeⅢ-WF polymer modified asphalt and its performances were redefined. This TypeⅢ-WF polymer modified asphalt, W denotes Water resistance and F denotes Flexibility, can be utilized both for the base course and the surface course of the orthotropic steel deck pavement. Recently this modified asphalt is utilized for the orthotropic steel deck pavement in Japan, although the most popular method is the Gussasphalt method. To apply this method in developing countries, contractors need to import this plant-mix modifier from Japan. But as this asphalt does not need any special machines, this TypeⅢ-WF polymer modified asphalt can be constructed in developing countries without any problems. But the problem is that the governmental agency tries to repave the bridge after 10 or 15 years, this modifier needs to be imported again from Japan. This modifier is not locally available.

In Table 6.2, the comparison of four pavement methods is shown. They are Gussasphalt method, Epoxy asphalt method, Type Ⅲ-WF polymer modified asphalt method and Proposed pavement method by GARBLT. In this table SMA pavement method is not included as SMA pavement is no longer utilized for the orthotropic

Table 6.2 Comparison of orthotropic steel deck pavement methods

No.	Type of Pavement	Comment	Cost	For developing countries
1. Gussasphalt		At present this is the most popular method in Japan. This method was applied for all of the long span bridges of the Honshu-Shikoku Bridge Expressway Company and showed the long time durability.	Most expensive.	Difficult to apply because special machines and TLA are needed. Repavement by local government is difficult.
2. Epoxy Asphalt		In USA, this method is the most popular. Experiences in Japan is only a few. For construction, special machines are not needed.	Cheaper than Gussasphalt method.	First application is possible but repavement is difficult.
3. Type Ⅲ-WF polymer modified asphalt		Only polymer modifier is needed. There are not many examples. For construction, special machines are not needed.	Cheaper than Gussasphalt method.	First application is possible but repavement is difficult. More suitable for developing countries than Epoxy asphalt.
4. Proposed pavement by GARBLT		First resin bonded surfacing is constructed. Over this layer, conventional dense graded asphalt is constructed. For construction, special machines are not needed. As there are no examples in Japan, some experiments are needed to decide the specification.	Probably cheapest	If the spec. is decided, this pavement can be constructed easily by local government. Most suitable for developing country.

steel deck pavement in Japan.

The Gussasphalt method is reliable but to construct this pavement in developing countries, special machines and special materials need to be imported. At least for the first application, Japanese contractors can construct this pavement. But the repavement by the local governments is very difficult.

To construct epoxy asphalt in developing countries, the necessary materials need to be imported from Japan. As only conventional paving machines are needed, the construction cost is cheaper than the Gussasphalt method. It may not be difficult for Japanese contractors to construct this pavement in developing countries. But the repavement by the local government may not be easy because the necessary materials need to be imported and the temperature control of construction work is difficult. Therefore this method may not be suitable for developing countries.

Type Ⅲ-WF polymer modified asphalt can be constructed in developing countries by the conventional paving machines. Only Type Ⅲ-WF polymer modifier needs to be imported from Japan. To repave the steel deck after about 10 years, only the modifier needs to be imported from Japan. Therefore this method is more suitable than the epoxy asphalt pavement method for developing countries. Also the cost of this method is cheaper than the Gussasphalt method.

The GARBLT proposed method may be the cheapest among four alternatives in Table 6.2. This method, if the specification is precisely decided, can be executed by the local governments of developing countries as shown by the Egyptian Government. Therefore the repavement is easy. This method is suitable not only for developing countries but also for Japan, as the construction cost is cheaper. But to adopt this method, in Japan, coal tar and chromium are hazardous to human health and are already banned. 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 epoxy. Some more experiments to confirm the effective resin bonded surfacing are needed.

If a new bridge is constructed in a developing country as a Japanese ODA project, the orthotropic steel deck is coated by inorganic Zinc-Rich paint at the shop. Then at the site, the mixture of modified epoxy and polyurethane will be applied and then the aggregate graded 2 to 7mm will be scattered to form a resin bonded surfacing. Then the dense graded asphalt will be applied. As this method has never been constructed in Japan, the pavement of this method needs to be constructed experimentally for the actual bridges and the contents of the polymer which fixes small aggregates need to be decided. The basic idea is to construct a resin bonded surfacing and over this surfacing, the asphalt concrete is overlaid so that the aggregates of the resin bonded surfacing will act as a "tooth" for the overlaid asphalt concrete. At the same time, the mixture of modified epoxy and polyurethane will protect the steel deck surface from the corrosion.

6.4 CONCLUSION

Based on the study results of Chapter 3, 4 and 5, the following design ideas are proposed in this chapter. They are as follows;

(1) For the long span bridges in developing countries, manually driven inspection vehicles are recommended because of the durability and the easiness of maintenance. When the manually driven inspection vehicles are designed, the both sided driving gears are recommended to cope with the inclination of the rails. Also the redundancy of the driving gears is very important so that even if some driving gears are broken during the long term of maintenance, the vehicles can be utilized. If the longitudinal gradient of the bridge girder exceeds more than 1%, the electrification of the inspection vehicles is needed. The countermeasures against the theft of electric parts are needed.

(2) A new cable corrosion protection system utilizing steel covers all along the main cables is proposed. This system should work well for more than 30 years in the tropical climate area similar to the climate around the Matadi Bridge. Also the maintenance of this system is easy because there are no electric parts. This

system is recommended in the tropical developing countries.

(3) The orthotropic steel deck pavement method proposed by GARBLT is compared with other pavement methods. It is concluded that if this method is improved to comply with the present environmental regulations, this method is recommended for developing countries which maintain long span bridges with orthotropic steel decks.

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6) For example, https://www.chemcosystems.com/epoxy_faqs/, “What are the key points to successful placement of epoxy asphalt concrete pavement?”

7) Follow-up cooperation study on the project for construction of the Suez Canal Bridge(pavement on steel deck) Final report, pp Japan International Cooperation Agency, Oriental Consultants Co. LTD, Chodai Co. LTD. Pp.5-3, March 2013 (in Japanese)

Chapter 7. CONCLUSION

For the design of long span bridges in developing countries, in addition to the conventional design method which is developed and utilized in developed countries, there are other design ideas which are specifically needed in developing countries. Various maintenance problems in Japan, which can be utilized for the future design, are reviewed first. Then three maintenance problems in developing countries are selected based on the experiences in Japan and taking into account the circumstances of developing countries. These three problems are analyzed and the countermeasures are planned. Based on this analysis, the better design ideas for the long span bridges in developing countries are proposed.

The three problems are, (1) the problems of maintenance facilities of suspension bridges, (2) the corrosion problem of main cables of suspension bridges and (3) the repavement problem of orthotropic steel deck.

For (1), the case of the Matadi Bridge in Democratic Republic of Congo (DRC hereinafter) is investigated. As this bridge has been maintained by OEBK (Organization pour L'Equipment de Banana et Kinshasa) for a long time, more than 30 years, many maintenance problems happened. But still maintenance vehicles could be utilized and the various maintenance works were carried out by utilizing these vehicles. This was partly because the inspection vehicles were not electrified and the possibility of theft of parts was low, although the staff of OEBK wanted to have the electrified maintenance vehicles.

The request of electrification of maintenance vehicles by OEBK were unfortunately rejected because the maintenance of electrified vehicles were difficult.

OEBK requested the side span under girder inspection vehicles because it was very difficult to inspect or to repaint the side spans because these spans are very high above the ground. The side span inspection vehicles had been omitted to reduce the initial cost. But for the maintenance work, these vehicles are indispensable and should have been prepared from the beginning.

Some of the manually driving gears of the under girder inspection vehicles and the inside girder inspection vehicles were deteriorated considerably and it was decided to replace them. There were some places on the rails of the under girder inspection vehicles, where the driving wheels detached from the surface of rails and it became very difficult for vehicles to pass through these places. The reason of these slippages was investigated and as the countermeasure, the both sided driving gear was proposed. Afterwards all of the driving gears of inspection vehicles were replaced by these both sided driving gears. With these driving gears, the slippage problem was solved at the same time the redundancy of the driving wheels becomes twice. Then even if one or two driving gears are broken, still the inspection vehicles can be utilized smoothly. From these facts, for the design of inspection vehicles of long span bridges in developing countries, the adoption of manually driven inspection vehicles and the both sided driving gears are recommended.

But if the longitudinal gradient of the bridge exceeds more than 1%, the adoption of the electrified inspection vehicles becomes necessary. In this case various countermeasures to prevent the theft and the vandalism becomes important, such as the arrangement of the security guards all day long, etc.

When a truck with crane to lift the cable inspection vehicles on the main cable, was selected, it became clear that the girder width of the Matadi Bridge was slightly too narrow to allow the free passage of cable inspection vehicles along the main cables. This girder width was adopted to reduce the initial construction cost. But from the point of view of maintenance, a slightly wider girder width should be better. The bridge designers should recognize this fact and balance the initial cost and the easiness of maintenance in the future to find the best solution.

For (2), the main cables of the Matadi Bridge was investigated. As the corrosion protection system of the main cables of this bridge is the same as that of the former group of suspension bridges of Honshu-Shikoku Bridge Expressway Company, i.e. the Innoshima Bridge, the Ohnaruto Bridge, the Ohshima Bridge, the Shimotsui-Seto Bridge, the Kita Bisan-Seto Bridge & the Minami Bisan-Seto Bridge, the corrosions of main cables were feared. The corrosion protection system of this bridge consisted of the wrapping paste and the wrapping wires. A part of the main cable was unwrapped and the wood wedges were driven into the cable from 8 directions to investigate the inside of the cable. Both the upstream cable and the downstream cable were investigated. The result of investigation of main cables showed that the main cables are rusty to some extent, but the progress of the deterioration was slower than other suspension bridges in Japan, probably due to the less number of occurrence of dew condensation at the site. But at two cable bands, red rusted wires were found and they could be seen from the cable band bottoms. Before the bottom opening of these two cable bands, the water retention was confirmed on these cable bands. The water retention happened because when OEBK repainted the whole bridge by themselves, they painted the bottom of the cable bands and closed the bottoms, although the bottoms should be kept open. But the importance of opening of cable band bottoms was not shown in the as built drawings. As the cables were rusty to some extent and there were two places of red rusted wires, it was judged necessary to adopt some countermeasures. As the wrapping wires and the wrapping paste existed and this combination confines the humidity inside of the cable, a practically only one solution was the adoption of the dry air injection system and the installation of this system was recommended.

At the center cable band, from which the water was well drained because an original pipe was remained, only white rusted wires were observed after about 30 years. This situation was almost the same as the wires of the cable band section of the Innoshima Bridge, when the cable was opened after 7 years of completion. This fact showed that the inside of the well drained cable bands, the wires could be kept healthy for a long time. From this fact, the steel cover cable corrosion system is proposed and recommended for the new bridges in developing countries, whose climate is similar to the area of the Matadi Bridge.

For (3), the repavement problem of the orthotropic steel deck, the case of the Suez Canal Bridge was investigated. SMA(Stone Mastic Asphalt) pavement was experimentally utilized for the orthotropic steel deck pavement in Japan at that time and the standard of SMA pavement for the orthotropic steel deck pavement was established. This SMA pavement was adopted for the Suez Canal Bridge in 2001, because the special machines needed for the Gussasphalt pavement were unavailable in developing countries. But within one year, hair cracks began to appear partly because of the overloading of vehicles whose axle weights sometimes exceeded 25t. Although the General Authority for Roads, Bridges and Land Transport (GARBLT) continuously maintained the pavement, the crack progressed considerably. During this period, the crack problems of SMA orthtropic steel deck pavement were reported in Japan and at around year 2007, SMA pavement was not adopted anymore for the orthotropic steel decks in Japan. In 2011, the condition of the pavement of this bridge was investigated by the Japanese team and confirmed the heavily cracked pavement. The team recommended the Gussasphalt method for the repavement as SMA pavement was no longer adopted in Japan. In 2016, GARBLT decided to repave the orthotropic steel deck of the Suez Canal Bridge by their own method, which had the record of more than 20 years' service without any major problems on one bridge in Cairo City. GARBLT repaved the Suez Canal Bridge in 2016 and this pavement works well at present. If this pavement is durable, this pavement method is beneficial and recommended for many developing countries. This pavement method consists of two layers. One is the resin bonded surfacing. The other is the normal dense graded asphalt pavement over the resin bonded surfacing. The dense graded asphalt pavement does not need any special machines. For the resin bonded surfacing, GARBLT utilized the zinc-chromate primer and tar epoxy. But these paints are prohibited in Japan as they are hazardous material to human health. To utilize this method, these paints need to be substituted by the safe materials.