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Planning and Construction of Large Diameter Submarine Pipeline between Singapore and Malaysia*

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During pipe-laying the pipeline profile was checked and controlled by divers and computer simulation. The submerged weight was monitored by computer simulation and measurement of the tensile force of the pulling wire during each pulling operation.

1 Introduction

A twin submarine water pipeline, which is presently the largest diameter submarine pipeline in the world, was constructed and laid beginning in March 1986, and completed in 18 months by the Engineering and Construction Division of Kawasaki Steel Corp.

This project was part of a plan to supply treated water to Singapore to meet the projected increase in water demand. The submarine portion involved laying two pipelines across the 1.6 km strait between Singapore and Malaysia.

Although the diameter of submarine pipelines for drinking water is generally only up to 1 000 mm, this submarine pipeline has a diameter of 2 200 mm, which is far larger than any previously constructed. It was therefore necessary to solve a number of technical problems including:

(1) Execution and quality control of a 238 mm thick concrete cladding of the pipes.
(2) Fabrication and transportation of very long, heavy pipe strings 45 m in length and weighing 270 t.
(3) Planning and execution of a bottom tow method for the very rigid piepline, and profile control during construction.

During construction, trial concrete cladding was carried out, equipment for fabrication and transportation of the pipe strings were developed, and the pipeline profile was monitored by computer simulation, which resulted in successful completion of the project in spite of the difficulties involved in its execution. This report describes the major elements in the design and construction of this large diameter submarine pipeline.

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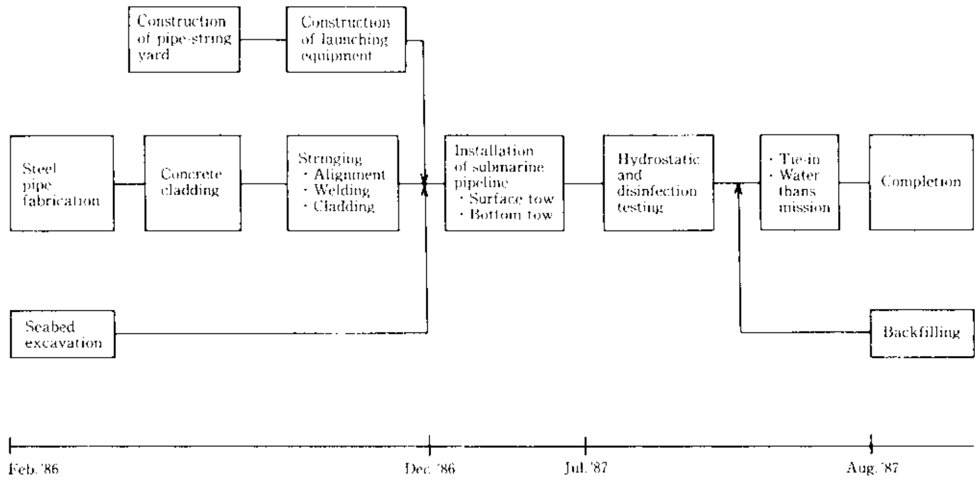

Fig. 1 Work flow and schedule

2 Outline of the Project

The pipe has an internal diameter of 2 200 mm with reinforced concrete cladding. The total length laid in the project was 2.5 km for each of the two lines.

The local contractor, a partner in the joint venture, was mainly responsible for civil work, e.g. seabed and land excavation and backfilling of the pipe, while Kawasaki Steel executed the pipelaying works, including the construction of temporary facilities.

2.1 Work Flow

The work flow outline of the project was as follows:

(1) Coated steel pipes 7.5 m long were delivered to the site, where a 5.6 m center section was clad with concrete.

(2) Units of six concrete-clad pipes were joined by welding to form 45 m long pipe strings, and corrosion protection and concrete cladding were applied to the joints.

(3) Two 45-m-long pipe strings were carried to the launch way and welded together to form a 90-m-long pipe string.

(4) The 90-m-long pipe string was pulled seaward, and the succeeding 90-m-long pipe string was welded and joined to the preceding string as stated in item (3) above.

(5) The operation items (3), (4) as stated above were repeated until the pipeline reached the designated position.

(6) After a hydrostatic test and disinfection, the pipeline was connected to existing pipelines.

Figure 1 illustrates the work flow outline for the project.

2.2 Work Specifications

A total of 5 500 t of steel pipe, 1 300 t of steel bar, and 9 600 m^3 of concrete was used in the project. **Table 1** shows the major work specifications.

Table 1 Work specifications (permanent work)

Item	Description	Quantity
Steel pipe	API 5L-X52 2 200 $\phi \times 19 t \times 7\,500 L$	5 500 t
Concrete cladding		
High yield deformed bar	16 mmϕ	1 300 t
Sulfate resistant cement	Grade 40	9 600 m^3
Joint protective coating	Heat shrinkable sheet	1 390 rolls
Seabed excavation	Marine clay, sand, rock	390 000 m^3
Backfilling	Sand	300 000 m^3

3 Concrete Cladding

3.1 Outline

Concrete cladding of the pipes was carried out to control the submerged weight and to reinforce the pipe and protect it from possible damage resulting from external forces and installation stresses.

Initially, 5.6 m lengths of the shop-fabricated 7.5 m length pipes were clad with concrete. After curing of the concrete, the clad pipes were transferred to the welding yard for joining by welding, and subsequently X-ray and hydrostatic-testing were accomplished. The joints were then wrapped with heat shrinkable polyethylene sheet for corrosion protection. After wrapping, the remaining 1.9-m-joint portions were clad with concrete.

The materials and application procedure were basically similar for the main pipes and joints.

Figure 2 shows the cross-section of the concrete-clad pipe. The material specifications for the reinforced concrete were as follows:

Cement : Sulphate-resistant cement
Water-cement ratio : 0.4 maximum
Coarse aggregate : 20 mm maximum size
Compressive strength : $\sigma_{28} = 400$ kg/cm^2 minimum

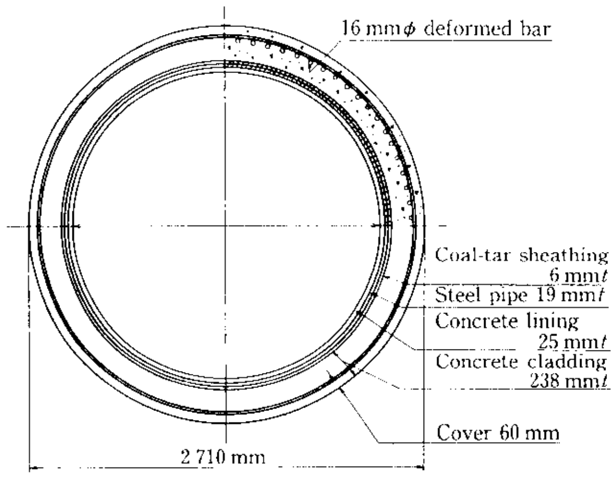

Fig. 2 Cross-section of concrete-clad pipe

Table 2 Results of trial casting

Item	Trial 1	Trial 2	Trial 3
Slump (mm)	180	220	155
Vibration time (min)	8	1	4
Pouring time (min)	25	15	60
Vibrator (kW × unit)	1.5×2	1.5×2	0.7×2
Surface finish	Bleed marks at top half	Air-holes at top half	Honeycomb
Concrete strength (kg/cm²)			
1 day	197	127	170
3 day	512	328	369
7 day	575	468	484
Concrete density (kg/m³)	2 412	2 379	2 439

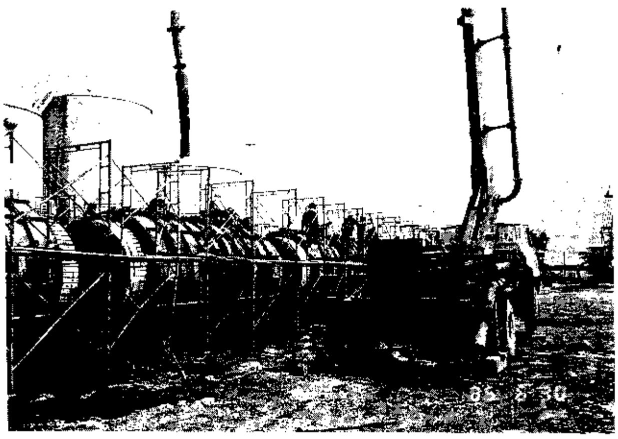

Photo 1 Concrete cladding

Reinforcing bar : 16 mmφ high yield deformed bar

The batch plant at the site was a dry-mixing type, and concrete was mixed in a 5.5 m³ capacity truck mixer. The form for placing the concrete was a circular horizontal-casting steel mold.

Concrete was placed in using a concrete pump through windows located on the sides of the mold to fill the bottom half, and then from top openings to cover the entire pipe. Vibration of the concrete was effected by external and poker-type vibrators.

Photo 1 shows a general view of the concrete cladding work.

3.2 Trial Casting

Prior to the actual pipe cladding, trial casting was carried out with 3 pieces of 2 200 mmφ × 2 400 mm length steel pipe in order to determine concrete characteristics, the capacity of vibrators, vibration time, and the surface finish of concrete cast with circular horizontal-casting steel molds. The results of the trial casts are shown in **Table 2**.

Based on these results, the following procedure was adopted:[1,2]

(1) Concrete Mix Proportion

- Sulphate-resistant cement : 380 kg/m³
- Fine aggregate (sand) : 800 kg/m³
- Coarse aggregate (20 mm): 1050 kg/m³
- Water : 152 l/m³ (W/C ratio = 0.4)
- Admixture (TB1-M Superplasticizer) : 3.8 l/m³ (1% of cement)
- Slump expected : 180 mm

(2) Procedure for Concrete Cladding

(a) Concrete was poured from the windows on the sides of the mold and the bottom-half vibrated with external vibrators attached to the mold, while concrete pouring was assisted by a poker vibrator.

(b) After concrete had covered half the pipe, the bottom-half vibrators were operated for 1 to 2 min.

(c) The side windows were then closed.

(d) Concrete was poured from the top openings, using poker vibrators, and the top-half also placed with external vibrators.

(e) After fully covering the pipe, all vibrators were turned on for an additional 1 min.

(f) One hour after pouring, the concrete was further vibrated for another minute to minimize bleeding or air pockets on the surface.

3.3 Concrete Quality Control

3.3.1 Concrete strength

The superplasticizer TB1-M, manufactured by Master Builders, was used in the early stages. It was then replaced by Conplast 430, which gave the better early strength development required for pipe transfer,

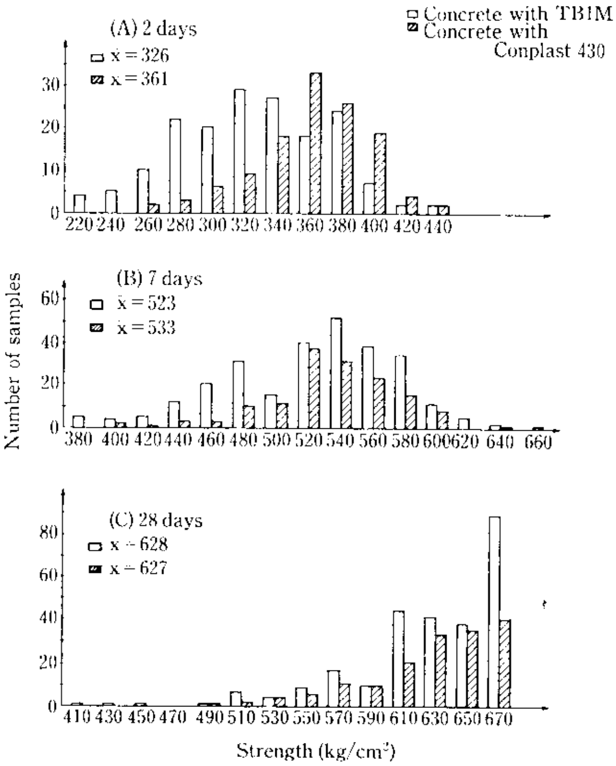

Fig. 3 Concrete cube strength with age using different admixture (TB1-M and Conplast 430)

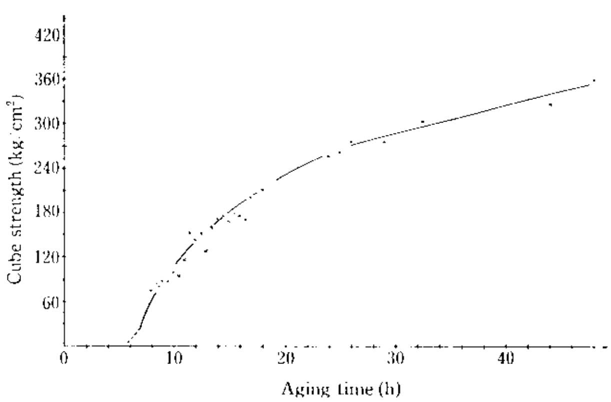

Fig. 4 Cube strength development with Conplast 430 by aging

and therefore reduced the construction cycle time.

With TB1-M, it took 16 to 20 h to obtain the concrete strength of 120 kg/cm^2 necessary for the transfer of the clad pipes, but only 10 to 12 h with Conplast 430.

Figure 3 illustrates the concrete strength achieved after curing of 2, 7, and 28 day respectively.

Figure 4 shows the progress of concrete strength development in the first 48 h using Conplast 430.

3.3.2 Weight of concrete clad pipe

Figure 5 shows in bar-chart form the weights of the concrete clad pipes.

The mean value of 599 pipes weighed was 37.19 t.

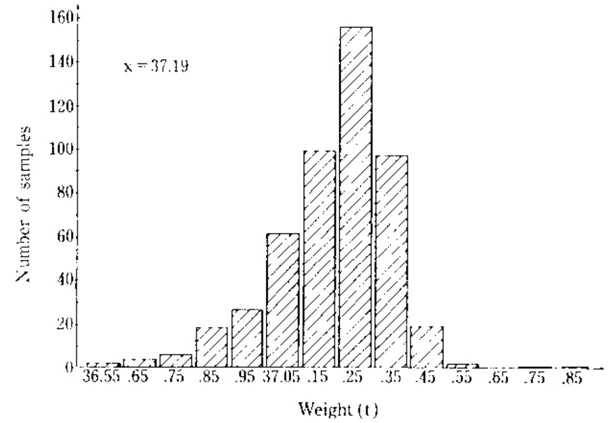

Fig. 5 Weight of individual concrete clad pipe

Table 3 Absorption test results

	Cube	Cylinder		
	Below surface 0.5 m deep	Below surface 0.5 m deep	At seabottom 15 m deep	
Soaking time (d)	30	31	28	50
Fresh water (%)	0.90	0.12	—	—
Seawater (%)	1.14	0.25	0.27	0.34

The mean value for the first 50 pipes however was 37.07 t, and 37.29 t for the last 50 pipes. It appeared that the steel molds, after more than 60 uses each, had deformed and expanded with repeated use, and hence the concrete circumference had enlarged slightly.

3.3.3 Water absorption of concrete

The water absorption ratio of the concrete was measured, since it definitely affects the submerged weight of the pipe.

Two types of specimen were used:

(1) 150 mm cubes, from which water penetrated from all six faces.

(2) 150 mmϕ × 238 mm height cylinders, which simulated the actual concrete-clad pipe. The cylinders were 238 mm in height, which equalled the thickness of the concrete cladding, and were sealed at the bottom and circumferential surfaces by the cylinder mold so that absorption would take place only at the top surface.

The results are tabulated in **Table 3**.

4 Fabrication and Movement of Pipe Strings

4.1 Outline

At the welding yard, six lengths of clad pipes were joined by welding to form 45-m-long pipe strings.

The completed pipe strings were moved to the launch

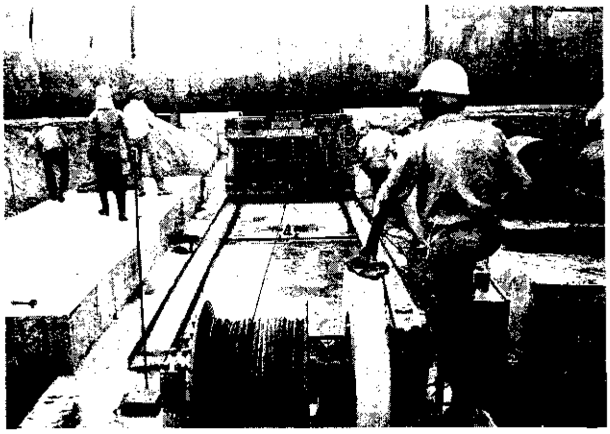

Photo 2 Pipe boggie

way using the following methods:

(1) Rolling on timber skids while being pulled by winch.

(2) Raising and transporting by pipe bogie with hydrau-lic jack. **Photo 2** shows the pipe bogie system.

4.2 Procedure

(1) Pipe Transfer to Welding Pit: The concrete-clad pipe was lifted by a 150-t crane and weighed, then placed in the welding pit.

(2) Alignment: Alignment of the joints was accomplished with two 30 t alignment jacks and an external clamp. The straightness of the pipe string was checked to a 3 mm/3 m tolerance by transit.

(3) Welding: The welding procedure and welders' qualification tests were carried out in accordance with API 1104. All joint welding was completed by qualified welders.

(4) Non-destructive Test: The welding was 100% tested by X-ray in accordance with API 1104.

(5) Hydrostatic Test: Each string was hydrotested at a pressure of 27 kg/cm^2 maintained for 24 h.

(6) Corrosion Protection Coating: Heat shrinkable sheets were used for corrosion protection wrapping of the joints. The wrapping was visually inspected and pinhole-tested with a holiday detector at 19 200 V. Peeling tests were made at random to check the adhesive strength of the wrapping.

(7) Internal Mortar Lining: The internal lining at the joints was applied by hand with two layers of non-shrink mortar.

(8) Concrete Cladding of Joints: The concrete cladding at the joints was substantially the same as the main pipe cladding.

(9) Movement of Pipe Strings: The pipe string yard comprised two sections, A-yard and B-yard, because of the limited space available. In A-yard, pipe strings were rolled by wire rope wound several times around the pipe and pulled with a winch. A-yard strings were pulled on rollers by winch to B-yard for transfer to the launch way. In B yard, pipe strings were carried by two pipe bogies, each equipped with four 60-t hydraulic jacks.

5 Launch Facilities

5.1 Launch Way

Based on a consideration of soil conditions at the site, the foreshore wall, floor base, and machinery foundations were constructed of reinforced concrete, and the side walls of steel soldier piles and timber lagging. The side and back slopes were stabilized with a mortar lining. **Photo 3** show a general view of the launch way.

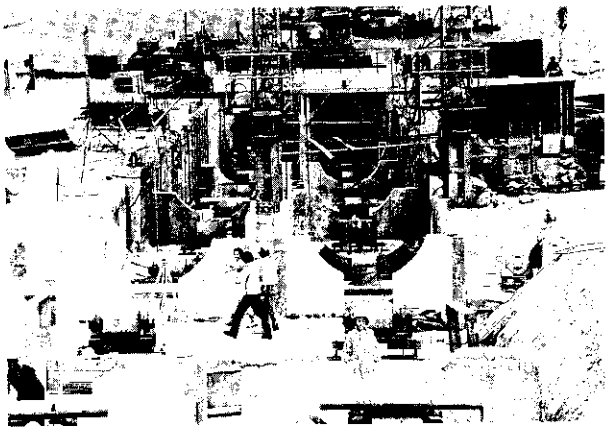

Photo 3 Launch way

5.2 Ramp Way

The ramp way structure is shown in **Fig. 6**. The ramp way consisted mainly of steel pipe piles and was cross-connected with H-beams by welding. The joints between the steel pipe pile and main girder, however, were made by the underwater junction method, which is shown in **Fig. 7**.[3,4] In rock areas, the steel pipe piles could not be driven and penetrated adequately and hence were inserted in pre-bored holes; underwater concrete was then poured into the holes to maintain the rigidity of the piles.

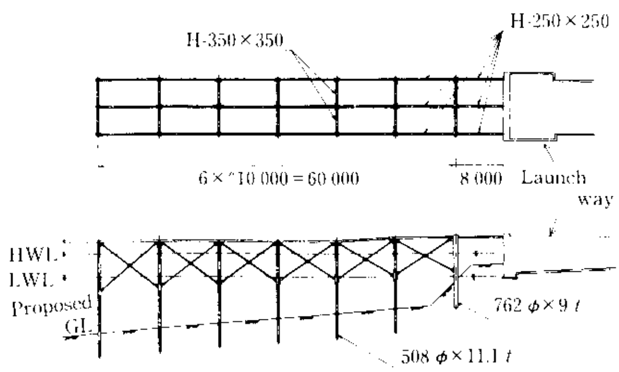

Fig. 6 Plan and side view of rampway

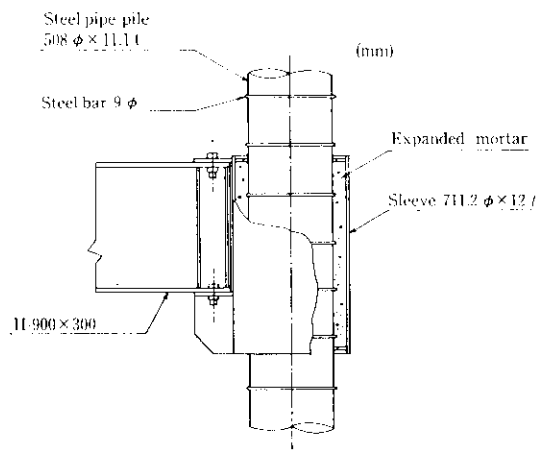

Fig. 7 Underwater junction method

Photo 4 Surface towing operation

6 Submarine Pipelaying and Design

6.1 Submarine Pipelaying Method

The surface and bottom tow methods are both considered suitable for a relatively short length underwater pipeline up to a few kilometers, such as that laid across a strait. For this project, the bottom tow method was adopted for the 2 200 m of continuous straight section, and the surface tow method for an approximately 180-m straight section ahead of the upstream bend in the pipeline. The surface tow method involves floating the pipeline at water surface level with buoyancy tanks. The pipeline is then towed by tug boats to the designated location and sunk into position.

The bottom tow method involves pulling the pipeline along the seabed or through a predredged trench with wire rope attached to the pipe head. Pulling is done by a winch anchored on land or on a barge. The method has been used on many occasions, but there had been very few cases in which such a large diameter pipeline had been laid by this method.

The important factors to be considered in using this method are the control of submerged pipe weight, pipeline profile, and pulling force.[5,6]

6.2 Surface Tow

Two strings approximately 180 m long each were assembled in the launch way. They were then slid down to the sea on the rollers by gravity and floated using a sufficient number of air-injected floater pipes. The pipe strings were separately towed on water surface by a 300-hp tug boat at the bow with a 80-hp tug boat at the stem to control skew. On reaching the designated location, the pipe strings were sunk into position by flooding floaters. It took 40 min to cross the 1.6 km of strait with each pipe string, and the deviation between the planned location and the actual pipe location was less than 10 cm. **Photo 4** shows a general view of the surface towing operation.

Prior to this operation, a computer simulation was carried out to predict the location and timing of positioning of the pipe string launch operation, based on tidal information. In addition, divers were sent down to check the position of pipe on the ramp way rollers, because the pipes had rotated and moved during the launching operation.[7]

6.3 Bottom Tow

Two 45-m-long pipe strings were joined into a 90-m length in the launch way and then alternately launched for the east and west lines by joining to the previously launched sections. **Figure 8** shows the concept for this operation. The bottom tow operation was a 24-h operation with labor working in alternating 12-h shifts.

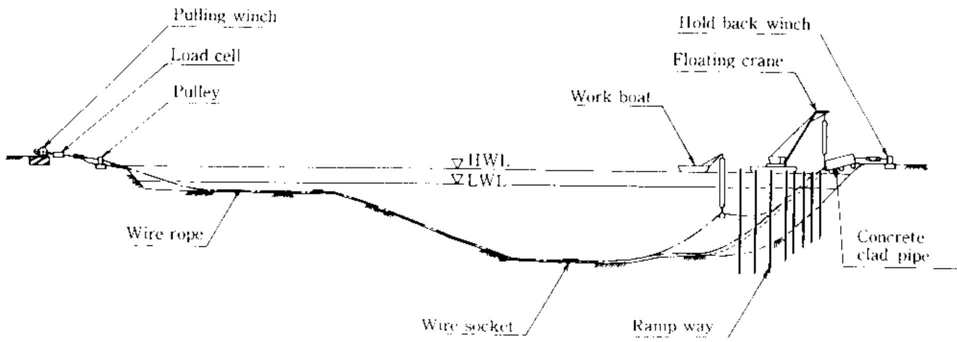

Fig. 8 Pulling plan

Before each pull commenced, the concrete had to develop a compressive strength greater than 120 kg/cm^2, as this was the estimated maximum pressure on the concrete of the launch way rollers. Quick setting concrete was tested for use but was not adopted because the setting time was not constant and very sensitive to ambient temperature and slight changes in the mix ratio. It took three welders 8 to 9 h to weld each joint in the launch way. Welding defects totalled 2% of the total welded length.

For the bottom tow operation, additional floaters were installed on the heavier pipe header to keep the header from sinking into the seabed. The movement of the pulling wire rope, the distance between the pipelines, and the seabed condition were routinely inspected by divers.

Launching speed was maintained at 1.6 m/min. with a 15-t hold-back winch. A 90-m length was therefore launched in about 1 h. **Photo 5** shows the launch way during the launching operation.

Pulling force was measured by tension meters attached to both the pulling wire rope and the hold-back wire rope. The pulling force at each pull was used to control the submerged weight for subsequent pulling so that the pulling force and the wire tension did not exceed the capacity of the winch and wire rope.[8]

After completion of the entire bottom tow operation, the riser pipe and 500-m-long suspended lengths supported by the ramp way rollers were lowered to the seabed by winch and wire rope after removal of the rollers and girders.

Photo 5 Lauching operation

On completion of the submarine pipelaying, the new line was tied in to pipe previously laid on both shores. Disinfection and hydrostatic testing of the completed pipelines were then carried out.

6.4 Pipe Profile Control and Design

It was necessary that the submarine pipelaying did not excessively stress the pipe under construction. This was checked using a personal computer in the site office.

The major input data were as follows:

(1) Pipe bending stiffness and weight (in air and sea water).
(2) Roller arrangement and specified trench level.
(3) Floater arrangement and buoyancy.
(4) Tide level.

The output data were as follows:

(1) Landing position of pipe on seabed.
(2) Pipeline profile for each stage.
(3) Bending stress on concrete and steel pipe for each stage.
(4) Minimum radius of curvature caused by bending.
(5) Reaction force of support rollers.

6.4.1 Analysis of pipe stiffness

As an analysis model, it was assumed that the concrete-clad pipe would behave as a composite beam of concrete and steel, and the tensile strength of the concrete was not considered. As a result, the neutral axis shifted from the central axis to the compressive side. (**Fig. 9**) The pipeline was analyzed as a continuous beam, considering cases when the pipe becomes detached from some of the support rollers before the pipe has reached the landing point as a result of the cantilevered front end. For the calculation of pipe stiffness, the mean static modulus of elasticity of concrete (E_C) and the mean compressive strength of concrete (σ_C) were tested and measured in accordance with British standards. The above results and the assumed values

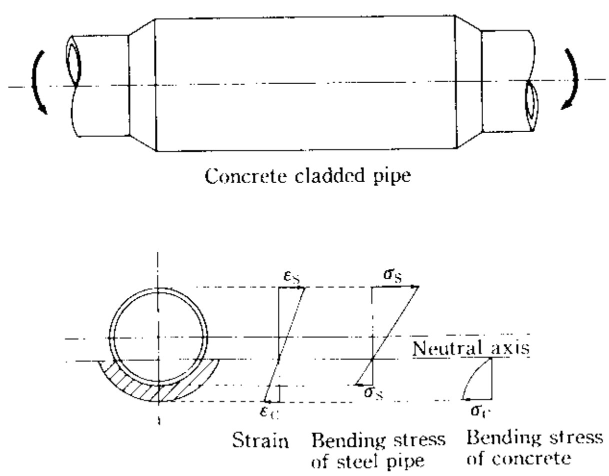

Fig. 9 Model of composite pipe

Table 4 Static mudulus of elasticity of concrete

Age (d)	Static modulus of elasticity, tested (kg/cm²)	Compressive Strength (kg/cm²)	Static modulus of elasticity, estimated by ACI code (kg/cm²)
7	4.49×10^5	570	3.67×10^5
28	4.54×10^5	705	4.08×10^5

Table 5 Bending stiffness (K) and moment of inertia (I)

	K (kg·cm²)	I (cm⁴)
Concrete cladded pipe	3.58×10^{13}	1.707×10^7
Steel pipe only	1.68×10^{13}	7.97×10^6

calculated in accordance with the American Concrete Institute (ACI) code are shown in **Table 4**.

The stiffness of concrete-clad pipe (K_B) calculated from the mean static modulus of elasticity and moment of inertia (I_B) are shown below. The stiffness and moment of inertia of the steel pipe itself are shown in **Table 5** for reference. The stiffness of the composite pipe (K_B) was calculated as follows:

$$K_B = K_S + K_C \quad \cdots(1)$$

$$K_S = E_S(I_S + I_{Sb}) \quad \cdots(2)$$

$$K_C = E_C I_C + E_S(A_S + A_{Sb})a^2 \quad \cdots(3)$$

where

K_S: stiffness of steel pipe and steel bar (kg·cm²)
K_C: stiffness of concrete (kg·cm²)
E_S: modulus of elasticity of steel (kg/cm²)
I_S: moment of inertia of steel pipe (cm⁴)
I_{Sb}: moment of inertia of steel bar (cm⁴)
E_C: modulus of elasticity of concrete (kg/cm²)
I_C: moment of inertia of concrete (cm⁴)
A_S: cross sectional area of steel pipe (cm²)
A_{Sb}: cross sectional area of steel bar (cm²)
a: distance between neutral axis and pipe central axis (cm)

$$I_C = \alpha\left[\frac{(r_C^4 + r_i^4)}{4} + a^2(r_C^2 - r_i^2)\right] + \sin 2\alpha \times \frac{r_C^4 - r_i^4}{8} - \frac{4}{3}a \sin\alpha \times (r_C^3 - r_i^3) \quad \cdots(4)$$

$$r_i = r_C - t_C/2 \quad \cdots(5)$$

$$\tan\alpha - \alpha - \frac{C_2}{C_1} = 0 \quad \cdots(6)$$

$$C_1 = r_C t_C \quad \cdots(7)$$

$$C_2 = \pi\frac{E_S}{E_C}(r_S t_S + r_{Sb} t_{Sb}) \quad \cdots(8)$$

where

r_C: radius to full concrete cladding (cm)

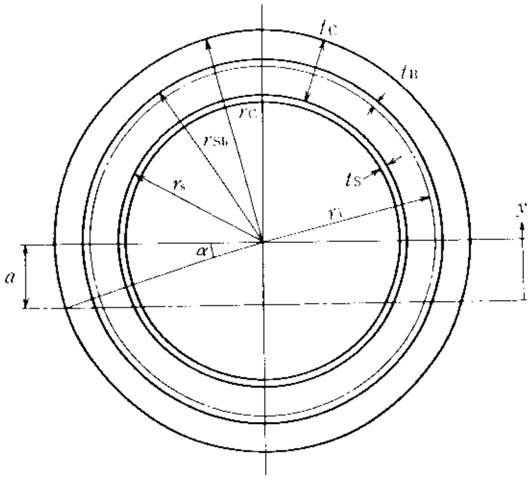

Fig. 10 Model for calculation of pipe bending stiffness

r_i: radius to centre of concrete cladding (cm)
t_C: thickness of concrete cladding (cm)
r_S: radius to center of steel pipe (cm)
t_S: wall thickness of steel pipe (cm)
r_{Sb}: radius to center of steel bar converted to ring section (cm)
t_{Sb}: thickness of steel bar converted to ring section (cm)

Figure 10 shows the model for the calculation of pipe bending stiffness.

6.4.2 Pipe weight

An estimated value for the weight of the concrete-clad pipe was calculated in the design stage. During construction, the actual concrete-clad pipe weight, water absorption ratio, and seawater density were measured.

Based on these data, the weight in air of the concrete-clad pipe and its submerged weight were determined to be as follows:

Weight in air = 6 131 kg/m
Submerged saturated weight = 236 kg/m
(including 10 kg/m as weight of absorbed water)
Buoyancy = 5 905 kg/m

where Weight of absorbed water = 0.24% × concrete weight
Seawater density = 1.024 t/m³

6.4.3 Support rollers

Roller positions were determined by roller bearing strength (90 t). Consideration was also given possible unevenness of the thickness of the concrete at the joints. The roller distances were as follows:

Launch way: 10 m spacing
Ramp way : 12.5 m spacing

The elevation of the rollers were based on computer simulation results.

6.4.4 Determination of pipeline profile during construction

The pipeline profile during construction was determined by computer simulation considering the following factors:

(1) Level of existing land and seabed
(2) Yard space for launch facilities
(3) Stiffness of submarine pipe
(4) Water depth
(5) Allowable stress on steel pipe

Based on the results of the computer simulation, a 3° gradient was specified for the launch way to maintain an allowable stress on the pipe and land the pipe at the nearest seabed landing point.

The over-bend system, which is usually adopted for small or medium diameter pipelines, could not be adopted for this pipeline because of its very high stiffness. Therefore, in the pipeline profile the pipe was moved out to sea at a 3° gradient and allowed to sag before reaching the landing point. The computer simulation showed that the landing point would be 550 m from the foreshore wall of the launch way. This was checked by divers and found to be consistent with the computer simulation estimates.

6.4.5 Comparison between computer simulation and actual profile of pipeline at each stage

There was some difference between the computer simulation and actual profile, which was attributable to the omission of the tensile strength of the concrete in the pipe stiffness calculation. When a 30% increase in pipe stiffness value was made to compensate for this, the simulated profile agreed reasonably well with the actual profile.

6.5 Submerged Weight Control and Pulling Force

The following factors were considered as affecting the pulling force:

(1) Seabed profile
(2) Friction between seabed and pipe
(3) Submerged weight of pipe and pulling wire rope

Although pulling force can be reduced by designing a lighter pipe, such pipe may be unstable during construction if it is too light. Therefore, the submerged weight of the heavy pipe was offset with floaters to obtain a net negative buoyancy of 35 kg/m in cohesive and 50 kg/m in sandy soils.

The estimated pulling force (F) was calculated using the following formula:

$$F = f_1 \times L_1 \times W_1 + f_2(L_2 W_1 + L_3 W_3) \quad \cdots(9)$$

f_1: friction coefficient between pipe and seabed (1.2)
L_1: pipe length in contact with seabed (m)
W_1: submerged unit weight of concrete-clad pipe with floaters (kg/m)
f_2: friction coefficient between pipe and rollers (0.01)

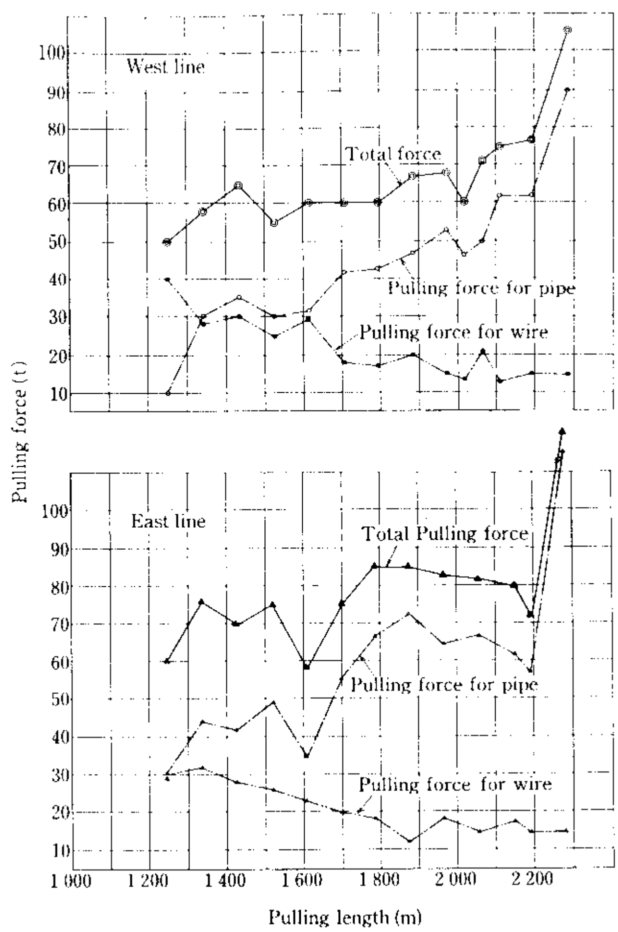

Fig. 11 Pulling force for pipe and wire with pulling length

L_2: pipe length supported by underwater rollers (m)
L_3: pipe length supported by on-land rollers (m)
W_3: unit weight of concrete-clad pipe in air (kg/m)

Based on this calculation, the pulling force was expected to be 66 t, and a 100-t capacity pulling winch was used. The actual range of pulling forces however was 50 to 130 t. **(Fig. 11)**

The probable reason for this difference was that the static friction of cohesive soil in the swampy area had a considerable inertial effect on the pipe.

7 Conclusions

The design and construction of the world's largest diameter submarine water pipeline, which was laid between Singapore and Malaysia, has been described. The report is summarized as follows:

(1) Trial casting was carried out for the concrete coating of a large diameter pipe at the site; based on the results, the concrete mix design and pouring procedure were determined, and the actual execution was satisfactory.
(2) The long and heavy pipe strings, which were 45 m

long and weighed 270 t, were fabricated using special alignment jacks and external clamps which were developed for this project. The pipe strings were moved using the following methods:

(a) Rolling on timber skids by winch

(b) Raising and transporting by pipe bogie with hydraulic jacks

(3) Both surface tow and the bottom tow methods were used. The pipe laying operation was analyzed by computer simulation and the pipeline was laid in the designated location within specified tolerances.

(4) In the design of the submarine pipeline, the stiffness of the composite pipe, which consisted of steel and reinforced concrete, was studied; and based on the data, the pipeline profile was controlled during construction.

This project was a pioneering one in that it involved the laying of a very heavy and large diameter submarine pipe. In spite of various difficulties, the project was successfully completed. With the experience gained, Kawasaki Steel Corp. is confident of successfully undertaking future projects of this magnitude and scope.

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