Appendix A Materials of Lectures and Seminars

A-6 Seminar for Wind Tunnel Test

Wind Tunnel Test for the Main Girder of Bago Bridge in Under Construction Stage and in After Completion Stage

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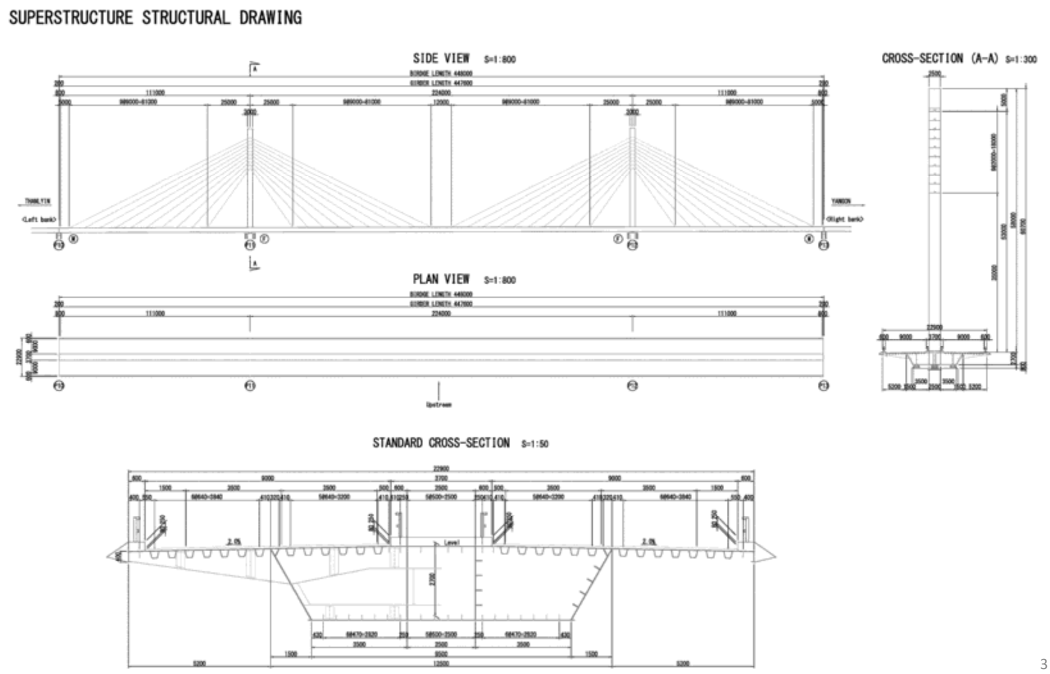

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Basic Condition to Evaluate Aerodynamic Stability

Elevation of main girder **17.645 m**
from the average water level (M.W.L.) to the upper surface level of the main girder
Girder width (*B*) **22.9 m**
Girder depth (*D*) **2.70 m** (B/D = 8.48)
Category of surface roughness: **II**
Power exponent of vertical profile of wind speed **0.16**
(Longitudinal) intensity of turbulence: **17 %** At the elevation of the main girder

<u>For after completion stage</u>
Basic wind speed (U_{10}) **30 m/s** 10 minute mean wind speed at 10m elevation
Design wind speed (U_d) **32.7 m/s**
At the girder elevation, the design wind speed U_d is $U_d = U_{10} \times E_1 = 30 \times 1.09 = 32.7$
<u>For under construction stage</u>
Basic wind speed (U_{10E}) **22.1 m/s**
non-exceeding prob. 0.6, construction period 3 yrs.
Design wind speed (U_{dE}) **24.1 m/s**
At the girder elevation ($22.1 \times E_1 = 24.1$).

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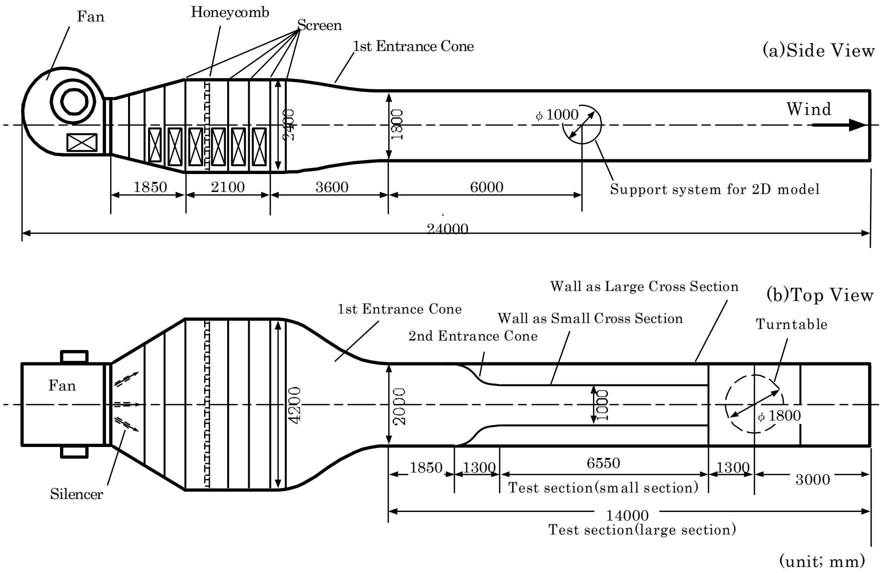
Wind tunnel in Kyoto University

Subject	Direction	Value
Wind Tunnel Type	-	Eiffel Type
Section Size	Width	1,000 (mm)
	Height	1,800 (mm)
	Length	6,550 (mm)
Wind Speed	Streamwise	0 ~ 25 (m/s)
Turbulence Intensity (in smooth flow)	Streamwise	< 0.5 (%)

Wind tunnel in Kyoto University



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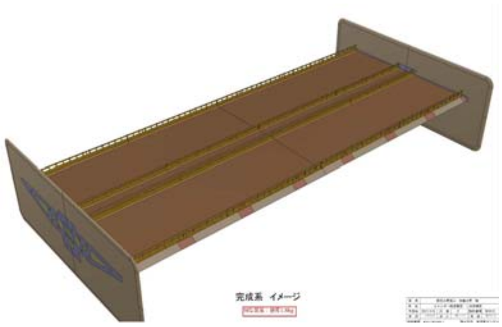

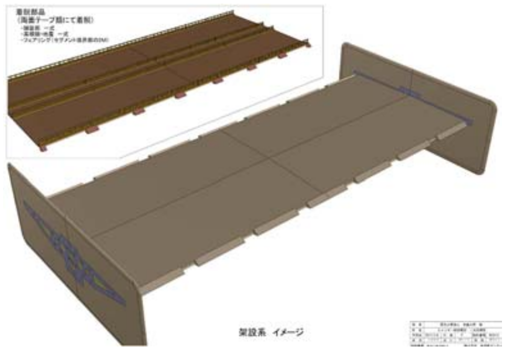

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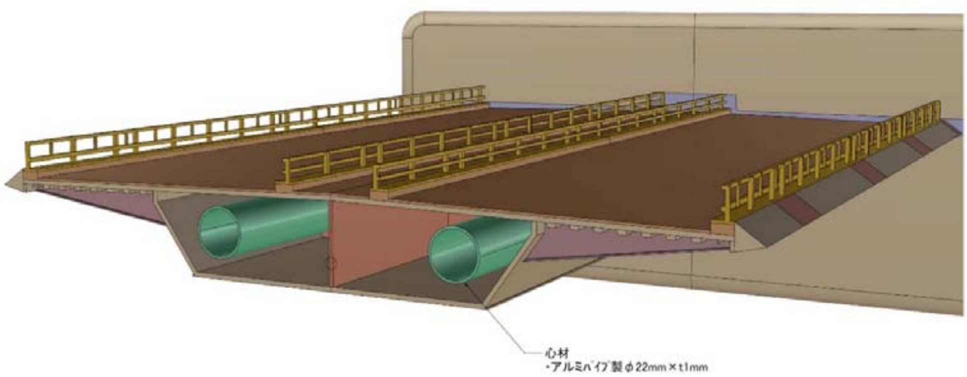

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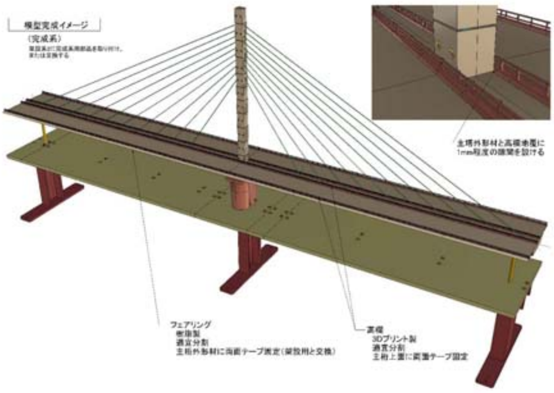

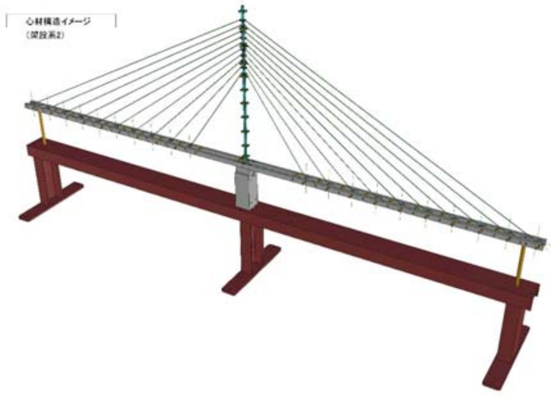

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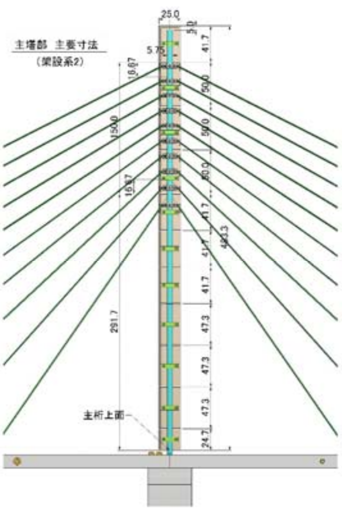
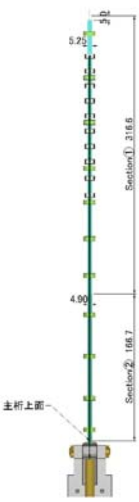
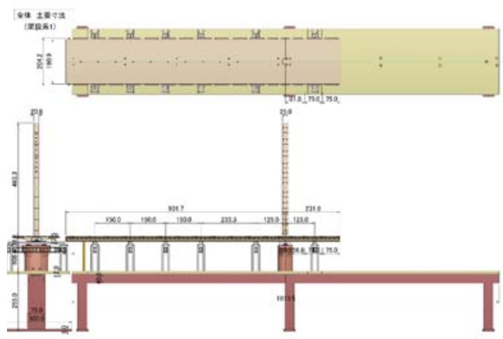
models







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Similarity rules

1) Wind velocity: $$\frac{U_p}{f_p L_p} = \frac{U_m}{f_m L_m} \quad (3.1)$$

2) Dynamic sensitivity (Scruton number): $$\frac{2m_p\delta_p}{\rho L_p^{\ 2}} = \frac{2m_m\delta_m}{\rho L_m^{\ 2}} \quad (3.2)$$

$$\frac{2I_p\delta_p}{\rho L_p^{\ 4}} = \frac{2I_m\delta_m}{\rho L_m^{\ 4}} \quad (3.3)$$

where, U : wind velocity (*m/s*),
f : natural frequency (*Hz*),
L : reference length (*m*),
m : mass per unit span length for heaving mode (*kg/m*),
I : moment of inertia per unit span length for torsional mode ($kg \cdot m^2/m$),
δ : logarithmic decrement of structural damping (-),
ρ : air density (kg/m^3) ,
$(\ )_p$: for real structure,
$(\ )_m$: for model

Note: Since the wind tunnel test was conducted in air, $\rho_p = \rho_m$.

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Similarity rules

item	unit	symbol: real structure	symbol: model	scale ratio	note
length	*m*	L_p	L_m	*n*	$L_p = nL_m$
time	*sec*	t_p	t_m	-	$\frac{U_p t_p}{L_p} = \frac{U_m t_m}{L_m}, \quad (t_p = t_m)$
wind velocity	*m/sec*	U_p	U_m	-	$\frac{U_p}{f_p L_p} = \frac{U_m}{f_m L_m}$
mass per unit span length	*kg/m*	m_p	m_m	n^2	$\frac{2m_p\delta_p}{\rho L_p^{\ 2}} = \frac{2m_m\delta_m}{\rho L_m^{\ 2}}$
moment of inertia per unit span length	kgm^2/m	I_p	I_m	n^4	$\frac{2I_p\delta_p}{\rho L_p^{\ 4}} = \frac{2I_m\delta_m}{\rho L_m^{\ 4}}$
natural frequency	*Hz*	f_p	f_m	-	-
logarithmic decrement of structural damping	-	δ_p	δ_m	1	$\delta_p = \delta_m$

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Equivalent mass

$$m_{eqz} = \frac{\int_{str} \left\{ m(s)\left[\phi_x^{\,2}(s)+\phi_y^{\,2}(s)+\phi_z^{\,2}(s)\right] + I_{rx}(s)\phi_{rx}^{\,2}(s) + I_{ry}(s)\phi_{ry}^{\,2}(s) + I_{rz}(s)\phi_{rz}^{\,2}(s) \right\} ds}{\int_{girder} \phi_z^{\,2}(x)dx} \tag{4.1}$$

$$I_{eqx} = \frac{\int_{str} \left\{ m(s)\left[\phi_x^{\,2}(s)+\phi_y^{\,2}(s)+\phi_z^{\,2}(s)\right] + I_{rx}(s)\phi_{rx}^{\,2}(s) + I_{ry}(s)\phi_{ry}^{\,2}(s) + I_{rz}(s)\phi_{rz}^{\,2}(s) \right\} ds}{\int_{girder} \phi_{rx}^{\,2}(x)dx} \tag{4.2}$$

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Equivalent mass

where. $\int_{str}\{\ \}ds$: integral along each structural member axis (s).

$\int_{girder}\{\ \}dx$: integral along girder axis (x).

$m(s)$: mass per unit span length (kg/m).

$I_{rx}(s)$, $I_{ry}(s)$, $I_{rz}(s)$: moment of inertia per unit span length around local x, y, z coordinates (kgm^2/m).

$\phi_x(s)$, $\phi_y(s)$, $\phi_z(s)$: displacement component of mode vector for local x, y, z coordinates.

$\phi_{rx}(s)$, $\phi_{ry}(s)$, $\phi_{rz}(s)$: rotational displacement component of mode vector for local x, y, z coordinates

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Target stage

For the under construction stage, the following 2 stages were focused:

- Before the lowest cable being installed and just after the first segment of the main girder was installed. (Heaving 1 DOF, This condition was abbreviated as **UC1**, hereafter.)
- Just before the last segment of the main girder in the main span is installed. (Heaving and torsional 2 DOF, **UC2**)

For after completion stage, the following modes combination was set to the model:

- (Heaving and torsional 2 DOF, **AC**)

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Target modes (UC1, Main girder)

Table 5.1.1 Dynamic properties of each mode (UC1, For main girder).

mode	natural frequency	equivalent mass m_{eqz}	equivalent moment of inertia I_{eqx}	mode shape
unit	Hz	kg/m	kg・m^2/m	-
1 and 2	0.655			tower bending (normal to cable plane) 1st
3 and 4	0.754			tower bending (along cable plane) 1st
5 and 6	2.594		5.875E+05	torsional (side span) 1st
7 and 8	3.977	1.171E+04		heaving (main span) 1st
9 and 10	4.199			tower bending (normal to cable plane) 2nd
11 and 12	4.810			tower bending (along cable plane) 2nd
13 and 14	5.197			torsional (side span) 2nd
15 and 16	5.826			torsional (main span) 1st
17 and 18	7.594			torsional (side span) 3rd
19 and 20	9.837			torsional (side span) 4th

Note: m_{eqz} is given by eq. (4.1) and I_{eqx} by eq. (4.2)

The shaded mode is the target mode.

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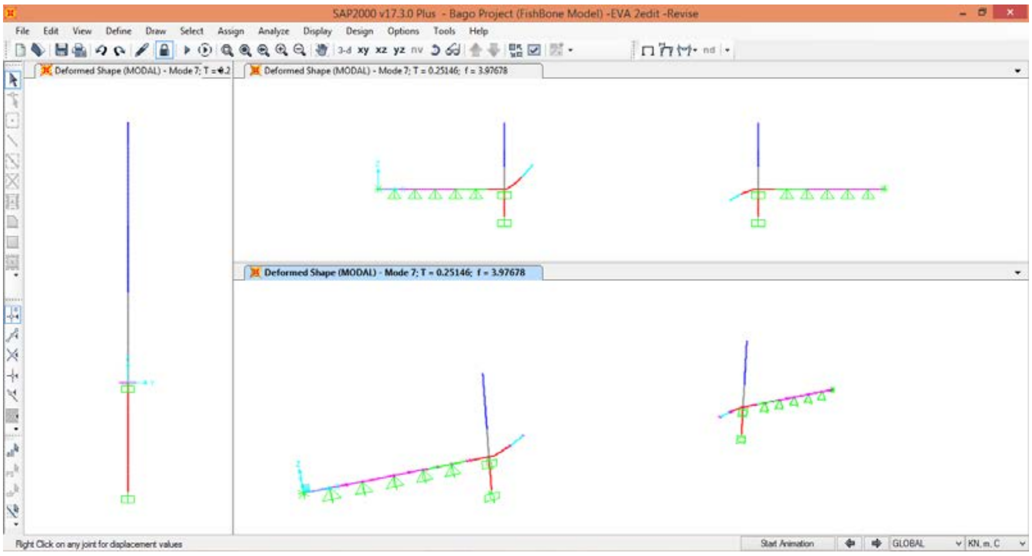
Target modes (UC1, Main girder)



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Target modes (UC2, Main girder)

Table 5.1.2 Dynamic properties of each mode (UC2, For main girder).

mode	natural frequency	equivalent mass m_{eqz}	equivalent moment of inertia I_{eqx}	mode shape
	Hz	kg/m	kg・m^2/m	
1 and 2	0.439	2.065E+04		heaving 1st
3 and 4	0.509			tower bending (normal to cable plane) 1st
5 and 6	0.822			heaving ?
7 and 8	0.944		8.855E+05	torsional (main span) 1st
9 and 10	1.000			heaving 2nd
11	1.137			torsional (side span) 1st
12 and 13	1.421			heaving 3rd
14	1.841			torsional (side span) 2nd
15 and 16	2.374			heaving 4th
17 and 18	2.816			torsional (main span) 2nd
19 and 20	2.859			heaving 5th

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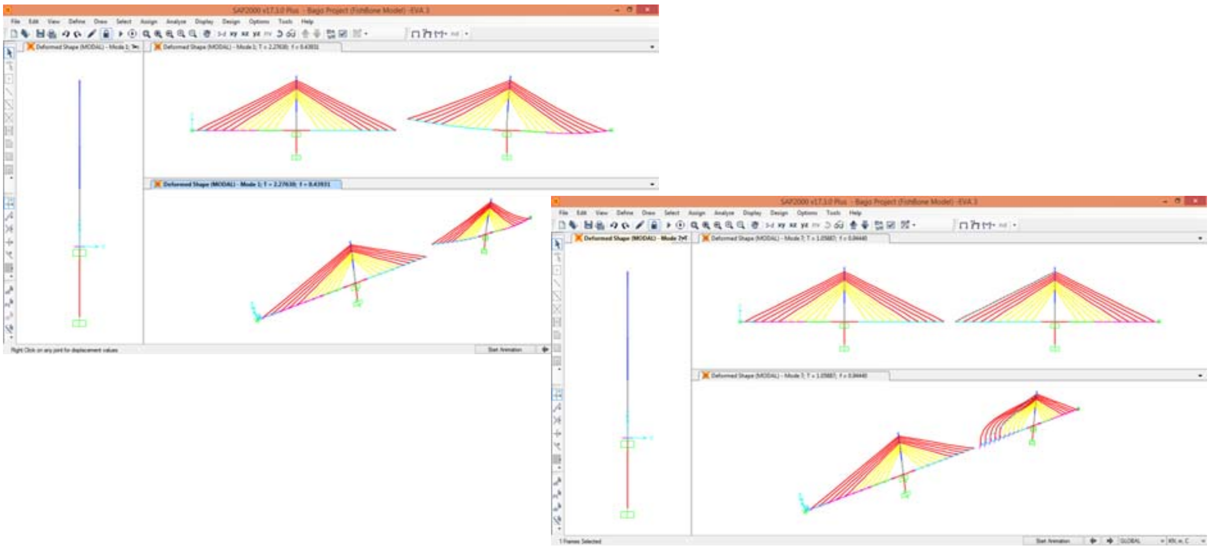
Target modes (UC2, Main girder)

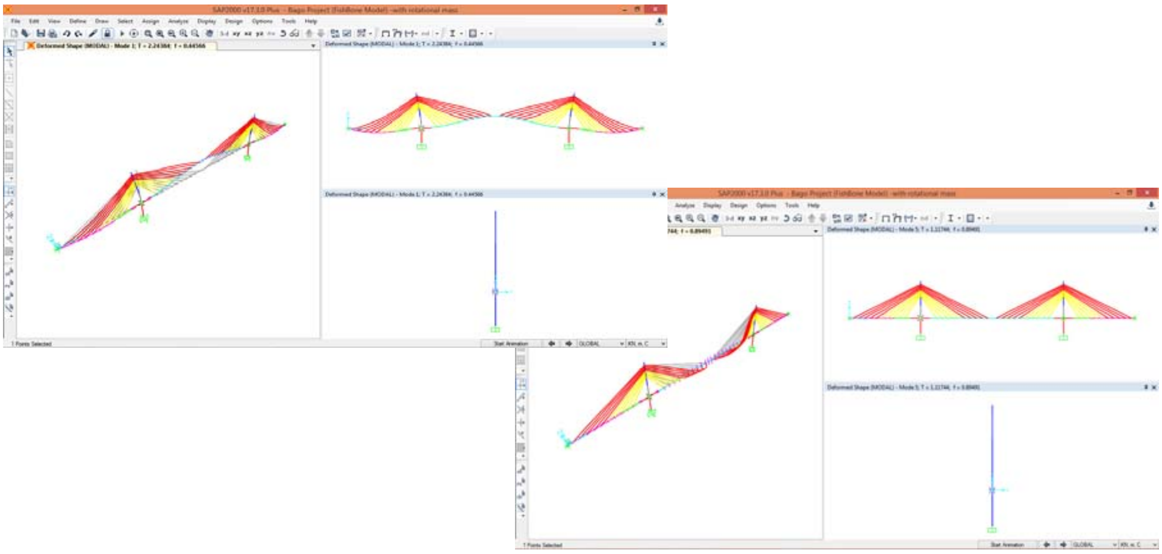


Target modes (AC, Main girder)

Table 5.1.3 Dynamic properties of each mode (AC, For main girder).

mode	natural frequency	equivalent mass m_{eqz}	equivalent moment of inertia I_{eqx}	mode shape
	Hz	kg/m	kg・m²/m	
1	0.446	2.044E+04		heaving 1st
2	0.509			tower bending (normal to cable plane) 1st (in-phase)
3	0.509			tower bending (normal to cable lane) 1st (out-phase)
4	0.740			heaving 2nd
5	0.895		8.749E+05	torsional (main span) 1st
6	0.984			sway 1st
7	1.014			heaving 3rd
8	1.137			torsional (side span) 1st
9	1.177			heaving 4th
10	1.583			heaving 5th

Target modes (AC, Main girder)

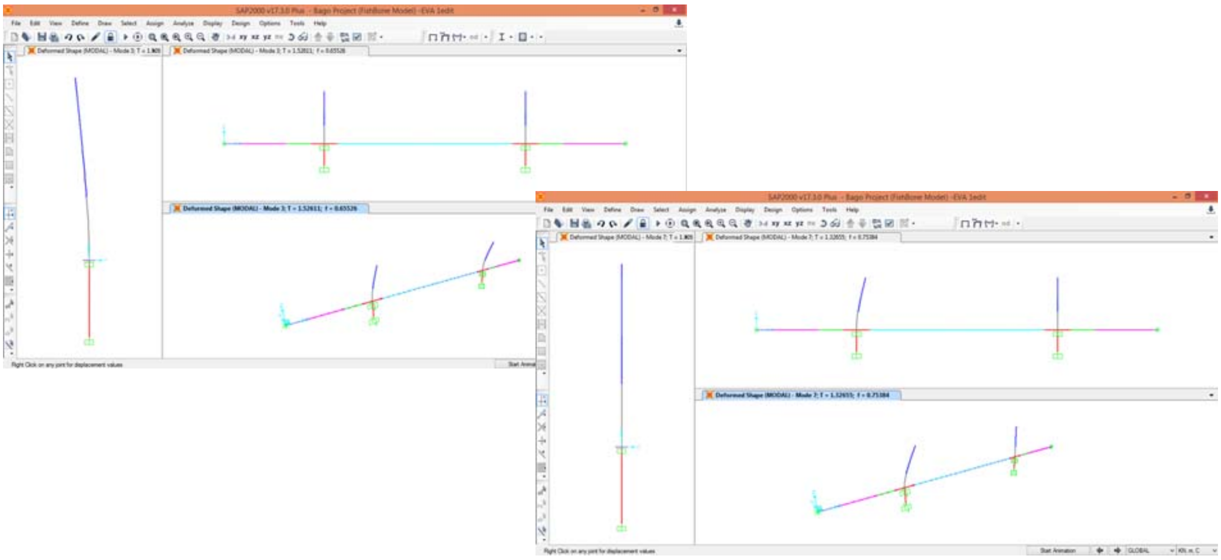


Target modes (UC1, Tower)

Table 5.2.1 Dynamic properties of each mode (UC1, For tower).

mode	natural frequency	equivalent mass m_{eqx}	equivalent mass m_{eqy}	mode shape
unit	Hz	kg/m	kg/m	-
1	0.234			
2	0.634			
3 and 4	0.655		1.224E+04	tower bending (normal to cable plane) 1st
5 and 6	0.680			girder heaving (side span) 1st
7 and 8	0.754	6.217E+03		tower bending (along cable plane) 1st

Target modes (UC1, Tower)

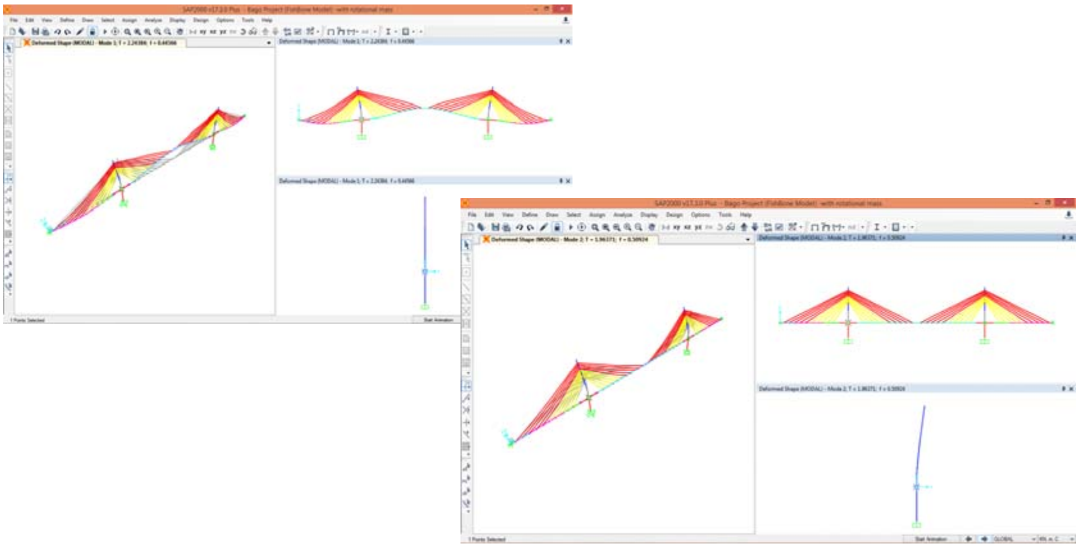


Target modes (AC, Tower)

Table 5.2.2 Dynamic properties of each mode (AC, For tower).

mode	natural frequency	equivalent mass m_{eqx}	equivalent mass m_{eqy}	mode shape
unit	Hz	kg/m	kg/m	-
1	0.446	7.393E+05		tower bending and girder heaving (along cable plane) 1st
2	0.509		1.949E+04	tower bending (normal to cable plane) 1st (in-phase)
3	0.509		1.949E+04	tower bending (normal to cable plane) 1st (out-phase)
4	0.740			tower bending and girder heaving (along cable plane) 2nd
5	0.895			girder torsional 1st
6	0.984			girder sway 1st

Target modes (AC, Tower)



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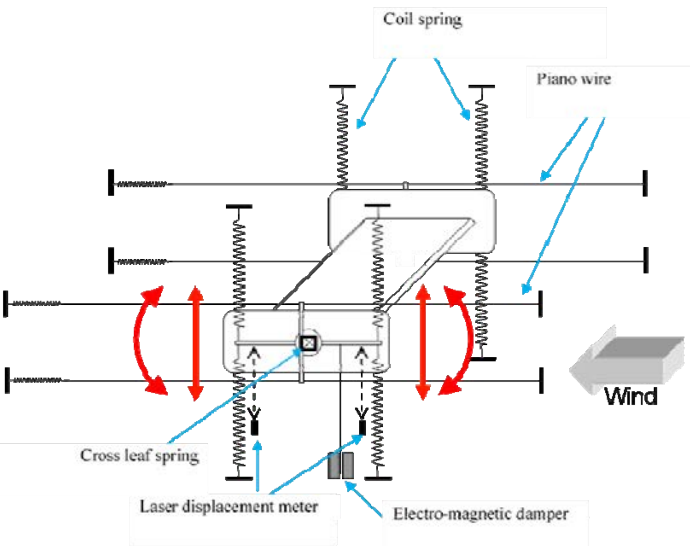
Test setup (Vibrational response measurement)



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Test setup
(Aerodynamic force
measurement)

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Free vibration test of main girder (UC1)

Table 6.2.1 Aerodynamic response of the main girder in UC1 (Heaving 1 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter	Corresponding figure
Smooth	0	o	o	Fig.6.2.1
	+3	o	o	Fig.6.2.2
	-3	o	o	Fig.6.2.3
Turbulent	0	o	o	Fig.6.2.4
	+3	o	o	Fig.6.2.5
	-3	o	o	Fig.6.2.6

“o” : The corresponding response was not observed.

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Free vibration test of main girder (UC2)

Table 6.2.2 Aerodynamic response of the main girder in UC2 (Heaving/torsional 2 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter	Corresponding figure
Smooth	0	o	o	Fig.6.2.7
	+3	o	o	Fig.6.2.8
	-3	o	o	Fig.6.2.9
Turbulent	0	o	o	Fig.6.2.10
	+3	o	o	Fig.6.2.11
	-3	o	o	Fig.6.2.12

"o" : The corresponding response was not observed.

Free vibration test of main girder (AC)

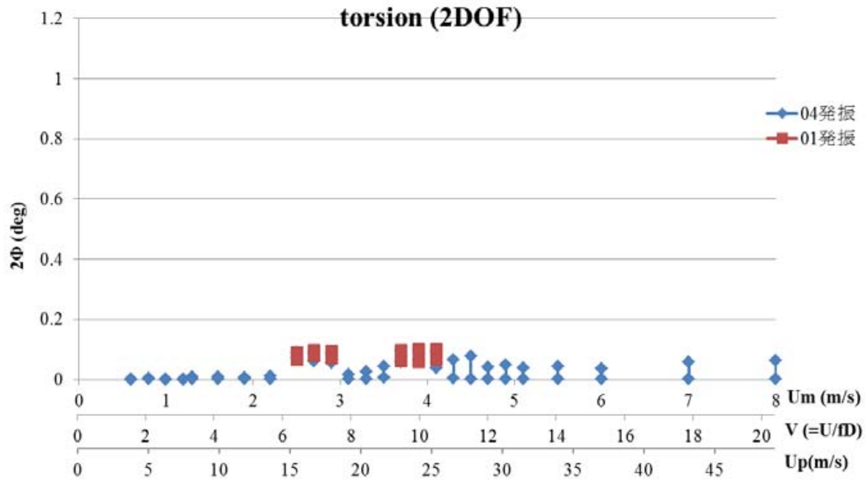
Table 6.2.3 Aerodynamic response of the main girder in AC (Heaving/torsional 2 DOF)

Flow condition	Vertical incidence angle of wind [deg]	Vortex-induced vibration	Flutter	Corresponding figure
Smooth	0	× Torsional (at around 21.6 [m/s])	o	Fig.6.2.13
	+3	× Torsional (15.4 - 17.9 [m/s]) (22.8 - 25.2 [m/s])	o	Fig.6.2.14
	-3	× Torsional (15.4 - 17.9 [m/s]) (20.3 - 24.0 [m/s])	o	Fig.6.2.15
Turbulent	0	o	o	Fig.6.2.16
	+3	o	o	Fig.6.2.17
	-3	o	o	Fig.6.2.18

"o" : The corresponding response was not observed.

"×" : The corresponding response occurred.

Free vibration test of main girder (AC)



Incidence angle of wind +3[deg.]
In smooth flow [Iu=11 [%])

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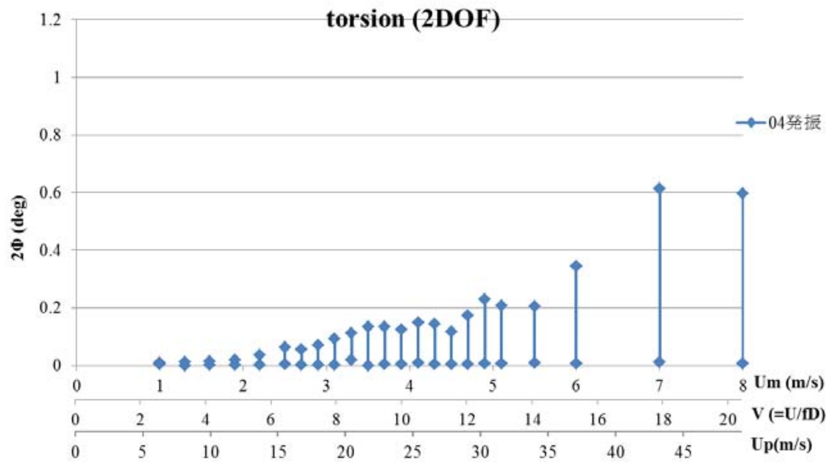
Free vibration test of main girder (AC)



Incidence angle of wind +3[deg.]
In turbulent flow [Iu=11 [%])

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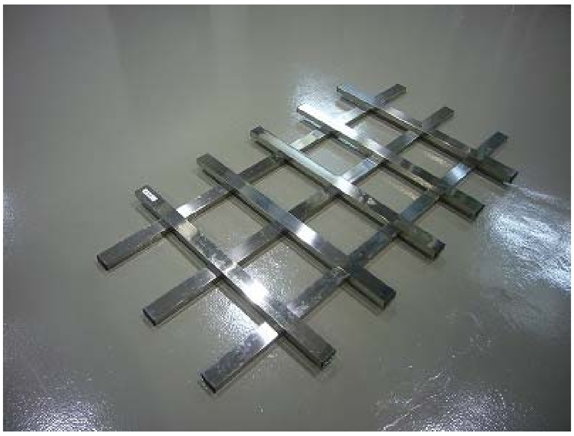
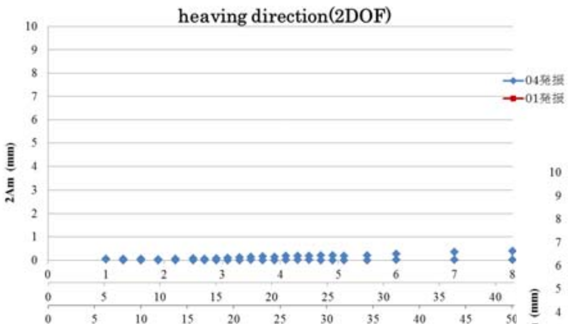
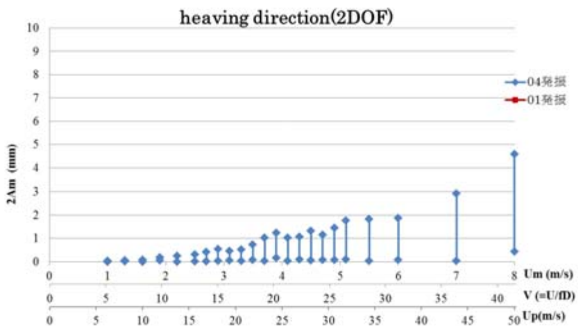
Grid to produce turbulent flow in wind tunnel



Free vibration test of main girder (AC)

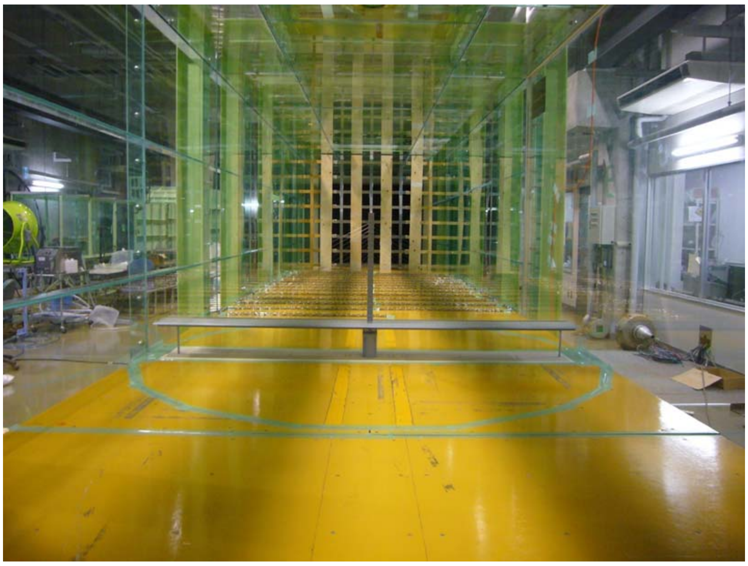


Incidence angle of wind 0[deg.]
In smooth flow



Incidence angle of wind 0[deg.]
In turbulent flow [Iu=11 [%])

Free vibration test of tower



Yawing angle: 90[deg.]
In turbulent boundary layer

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Free vibration test of tower (UC1)

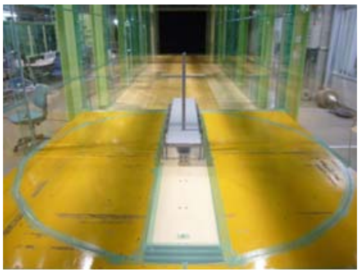

Table 6.3.1 Under construction 1 (UC1, Original tower configuration (without aerodynamic device))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	× in y-direction (16.5m/s~18.6m/s)	o	Fig.6.3.2
	5	× in y-direction (16.5m/s~18.6m/s)	× in y-direction (60.0m/s~)	Fig.6.3.3
	22.5	o	o	Fig.6.3.4
	45	o	o	Fig.6.3.5
	67.5	o	o	Fig.6.3.6
	80	× in x-direction (14.2m/s~22.3m/s)	× in x-direction (58.7m/s~)	Fig.6.3.7
	85	× in x-direction (14.2m/s~20.3m/s)	o	Fig.6.3.8
	90	× in x-direction (16.2m/s~22.3m/s)	× in x-direction (43.6m/s~)	Fig.6.3.9

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.

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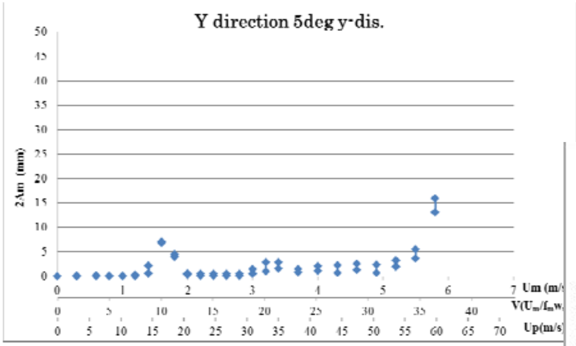
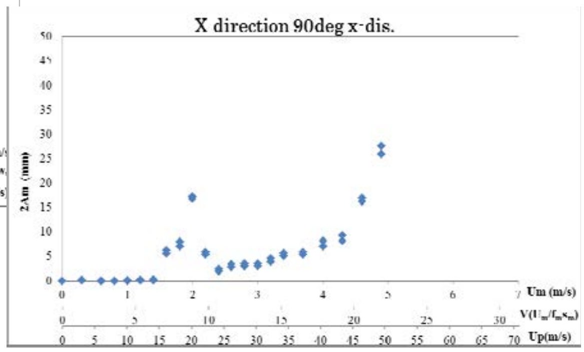
Free vibration test of tower (UC1)



Y direction: normal to cable plane
In smooth flow



X direction: along cable plane
In smooth flow

Free vibration test of tower (UC1)

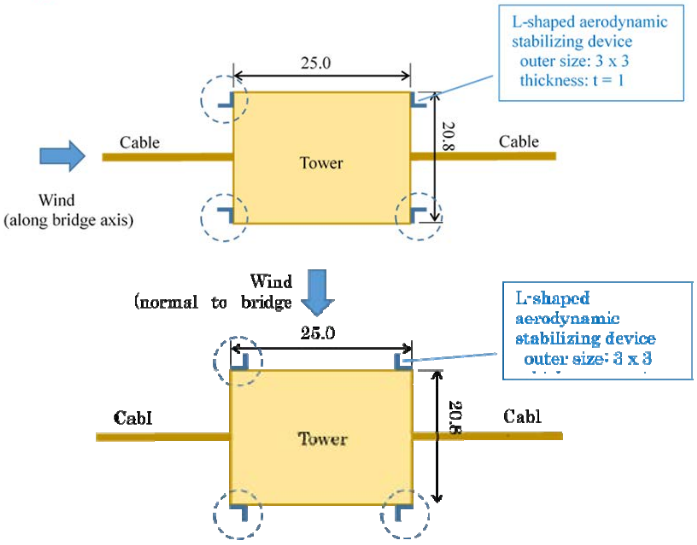
(With L-shaped aerodynamic device, length: 91.7mm (11.0m in real bridge))

Table 6.3.2 Under construction 1 (UC1, With L-shaped aerodynamic device)
(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	× in y-direction (12.6m/s~14.7m/s)	o	Fig.6.3.11
	90	× in x-direction (18.5m/s~20.6m/s)	o	Fig.6.3.12
Turbulent	80	o	o	Fig.6.3.13
	85	o	o	Fig.6.3.14
	90	o	o	Fig.6.3.15
	180	o	o	Fig.6.3.16

“o” : The corresponding response was not observed.
“×” : The corresponding response occurred.

Free vibration test of tower (UC1)
L-shaped aerodynamic device



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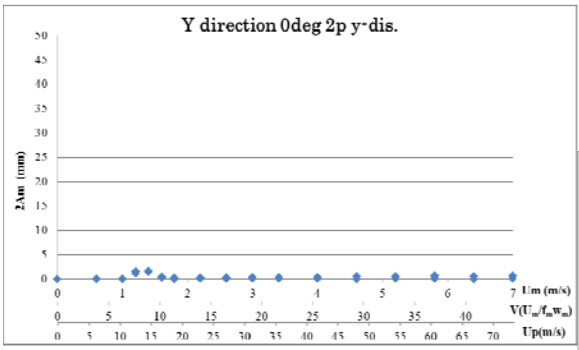
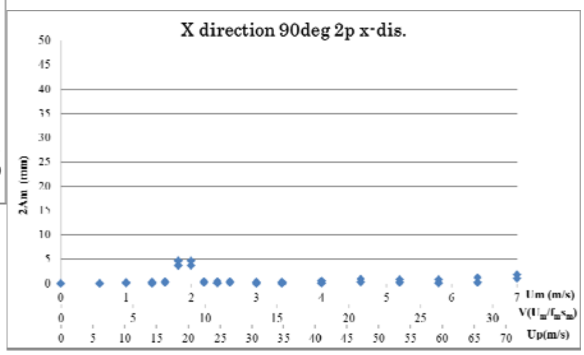
Free vibration test of tower (UC1), L-shaped aerodynamic device



Y direction: normal to cable plane
In smooth flow



X direction: along cable plane
In smooth flow

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Free vibration test of tower (UC2), L-shaped aerodynamic device

Table 6.3.3 Under construction 2 (UC2, With L-shaped aerodynamic device)
(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	× in y-direction (11.1m/s~29.5m/s) (34.2m/s~34.2m/s)	o	Fig.6.3.17
	5	o	o	Fig.6.3.18
Turbulent	0	o	o	Fig.6.3.19
	5	o	o	Fig.6.3.20

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.

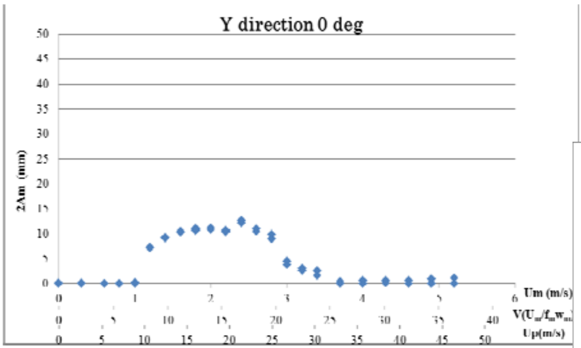
Free vibration test of tower (AC), Original configuration

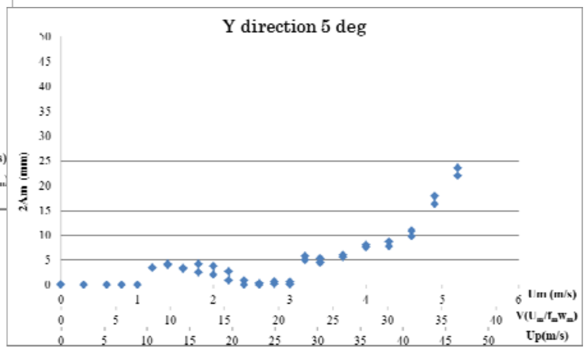
Table 6.3.4 After completion (AC, Original tower configuration)

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	× in y-direction (10.7m/s~30.4m/s)	o	Fig.6.3.21
	5	× in y-direction (10.7m/s~19.7m/s)	× in y-direction (28.6m/s~)	Fig.6.3.22
	10	o	o	Fig.6.3.23
	22.5	o	o	Fig.6.3.24
	45	o	o	Fig.6.3.25
	67.5	o	o	Fig.6.3.26
	90	o	o	Fig.6.3.27
Turbulent	0	× in y-direction (17.9m/s~19.7m/s)	× in y-direction (23.3m/s~)	Fig.6.3.28
	5	o	o	Fig.6.3.29
	10	o	o	Fig.6.3.30
	22.5	o	o	Fig.6.3.31
	45	o	o	Fig.6.3.32
	67.5	o	o	Fig.6.3.33
	90	o	o	Fig.6.3.34

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.

Free vibration test of tower (AC)





Y direction: normal to cable plane
In smooth flow

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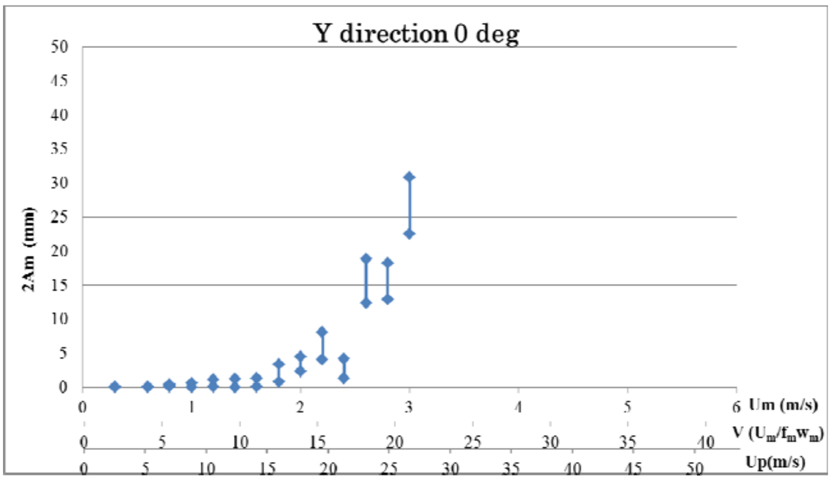
Free vibration test of tower (AC)



Y direction: normal to cable plane
In turbulent boundary layer

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Free vibration test of tower (AC),
L-shaped aerodynamic device, length: 91.7mm (11.0m in real bridge)

Table 6.3.5 After completion (AC, With L-shaped aerodynamic device)
(Length of aerodynamic device: 91.7mm (= 11.0m for real bridge) from the top of the tower))

Flow condition	Yawing angle [deg]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	× in y-direction (22.1m/s~25.8m/s)	o	Fig.6.3.35
	5	o	o	Fig.6.3.36
Turbulent	0	o	o	Fig.6.3.37
	5	o	o	Fig.6.3.38

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.

Free vibration test of tower (AC),
L-shaped aerodynamic device,
length: variable

Table 6.3.6 After completion (AC, With L-shaped aerodynamic device)

Flow condition	Yawing angle [deg]	Length of aerodynamic device [mm]	Vortex-induced vibration	Galloping	Corresponding figure
Smooth	0	141.7	× in y-direction (15.0m/s~40.0m/s)	o	Fig.6.3.39
	5	141.7	o	o	Fig.6.3.40
Turbulent	0	41.7	o	o	Fig.6.3.41
		141.7	o	o	Fig.6.3.42
		191.7	o	o	Fig.6.3.43
		233.4	o	o	Fig.6.3.44
	5	41.7	o	o	Fig.6.3.45
		141.7	o	o	Fig.6.3.46
		191.7	o	o	Fig.6.3.47
		233.4	o	o	Fig.6.3.48

"o" : The corresponding response was not observed.
"×" : The corresponding response occurred.

Aerodynamic derivatives

For a body moving in heaving/torsional 2DOF under the approaching wind speed U, the motion-induced lift force and pitching moment is expressed as:

$$
\begin{aligned}
L &= \frac{1}{2}\rho(2b)U^2\left\{kH_1^*\frac{\dot{\eta}}{U}+kH_2^*\frac{b\dot{\phi}}{U}+k^2H_3^*\phi+k^2H_4^*\frac{\eta}{b}\right\} \\
M &= \frac{1}{2}\rho(2b^2)U^2\left\{kA_1^*\frac{\dot{\eta}}{U}+kA_2^*\frac{b\dot{\phi}}{U}+k^2A_3^*\phi+k^2A_4^*\frac{\eta}{b}\right\}
\end{aligned}
\qquad (6.4.1)
$$

Where, b: half chord length [m] (b=B/2), η: heaving displacement [m] (positive downward), ϕ: torsional displacement [rad] (positive nose-up), k: reduced frequency ($k=b\omega/U$), ω: circular frequency of the motion, H_i^*, A_i^*, $(i=1,\cdots,4)$: aerodynamic derivatives.

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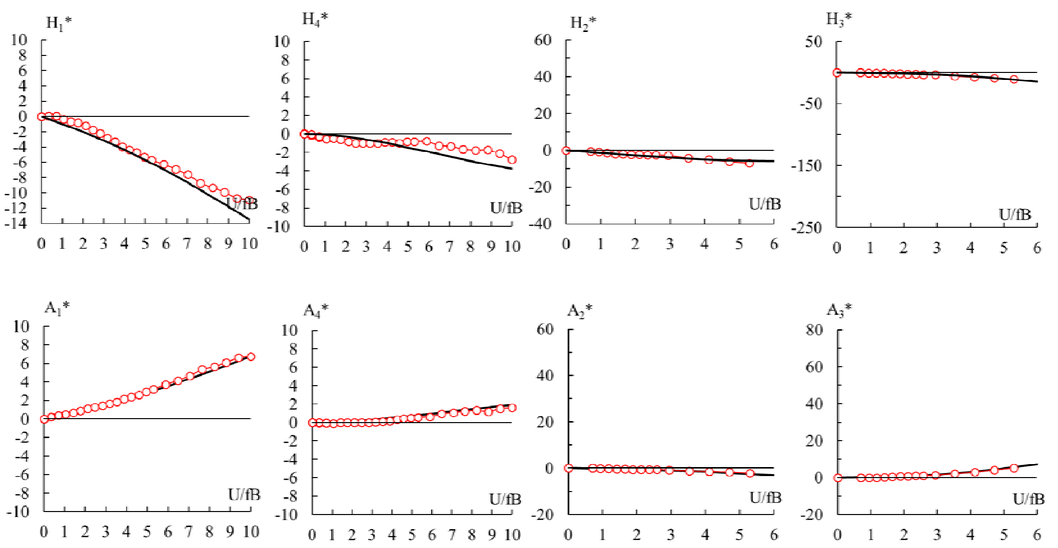
Aerodynamic derivatives of Main girder

Incidence of wind: 0[deg.]
In smooth flow



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Aero-static force components of Main girder

Drag force coefficient:

$$C_D = \frac{drag \quad force}{\frac{1}{2}\rho U^2 DL} \qquad (6.5.1)$$

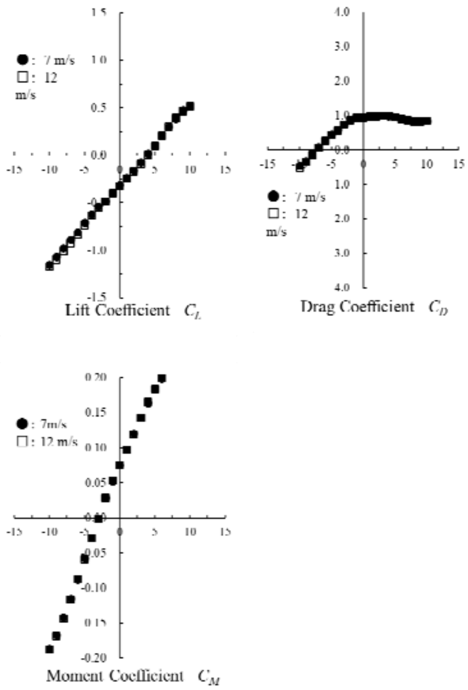
Lift force coefficient:

$$C_L = \frac{lift \quad force}{\frac{1}{2}\rho U^2 BL} \qquad (6.5.2)$$

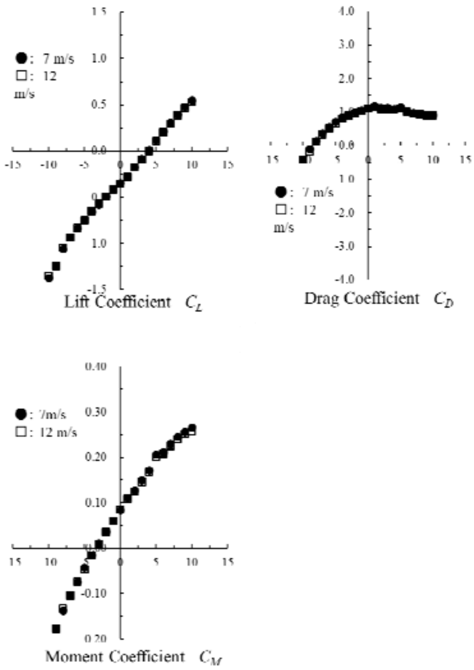
Pitching moment:

$$C_M = \frac{pitching \quad moment}{\frac{1}{2}\rho U^2 B^2 L} \qquad (6.5.3)$$

Aero-static force components of Main girder (UC) (In smooth flow)



Aero-static force
components
of Main girder (UC)
(In turbulent flow)



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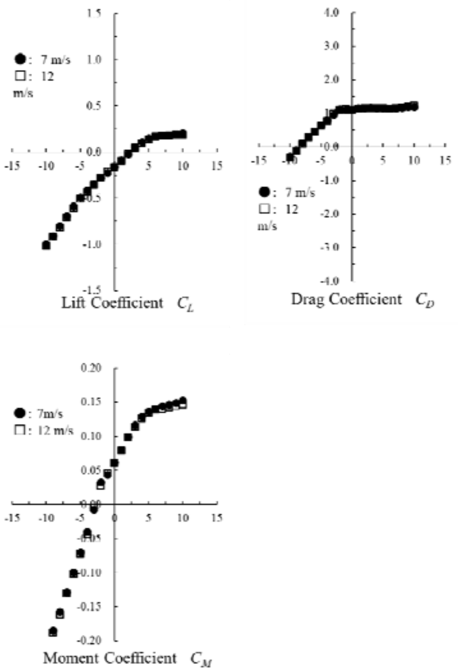
Aero-static force
components
of Main girder (AC)
(In smooth flow)



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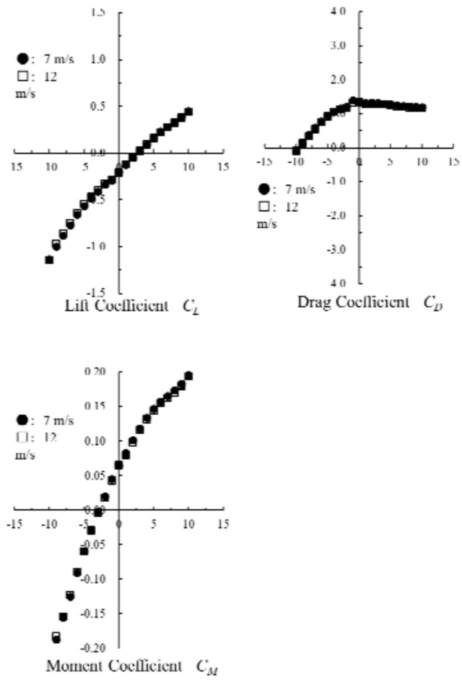
Aero-static force components of Main girder (AC) (In turbulent flow)



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Statistical analysis of wind speed record

The non-exceeding probability of yearly maximum value of 10-minutes mean wind speed x is described as:

$$\Pr\left[x_{\max}\big|_{year} < x^* \right] = \left\{ P(x^*) \right\}^N \quad (7.1)$$

where, $x_{\max}\big|_{year}$: yearly maximum of x, $P(x)$: non-exceeding probability of the wind speed x that is observed with a certain interval (Probability density function of $P(x)$, $p(x) = \dfrac{dP(x)}{dx}$, is sometimes called as mother distribution.), N: number of observation in a year (If the interval is one hour, N=24*365=8760.).

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Statistical analysis of wind speed record

Fischer-Tippet type I (Gumbel distribution):

$$\Pr\left[x_{\max}\big|_{year} < x^*\right] = \exp\left[-\exp\left\{a\left(x_{\max}\big|_{year} - b\right)\right\}\right] \equiv F_I\left(x_{\max}\big|_{year}\right) \quad (7.2)$$

The upper tail is expressed as:

$$P(x) = 1 - \exp\{-g(x)\} \quad (7.3)$$

In general, he non-exceeding probability of the extrema (yearly maximum) of the 10 minutes mean wind speed in Japan is well expressed by the Gumbel distribution as shown in eq.(7.2).

The parameters a and b are determined by the following formula:

$$a = \frac{\pi}{\sqrt{6}\sigma_x} = \frac{1}{0.780\sigma_x} \quad (7.10)$$

$$b = \bar{x} - 0.450\sigma_x \quad (7.11)$$

where, σ_x : standard deviation of x, $\bar{x}$: average of x.

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Statistical analysis of wind speed record

The expected maximum wind speed for a given return period R is determined as follows:
By taking logarithm of times to both sides in eq.(7.2),

$$\ln\left[-\ln\left\{\Pr\left[x_{\max}\big|_{year} < x^*\right]\right\}\right] = -a\left[x_{\max}\big|_{year} - b\right] \quad (7.12)$$

The value of $\Pr\left[x_{\max}\big|_{year} < x^*\right]$ is related to R as shown below:

$$R = \left\{1 - \Pr\left[x_{\max}\big|_{year}\right] < x^*\right\}^{-1} \quad (7.13)$$

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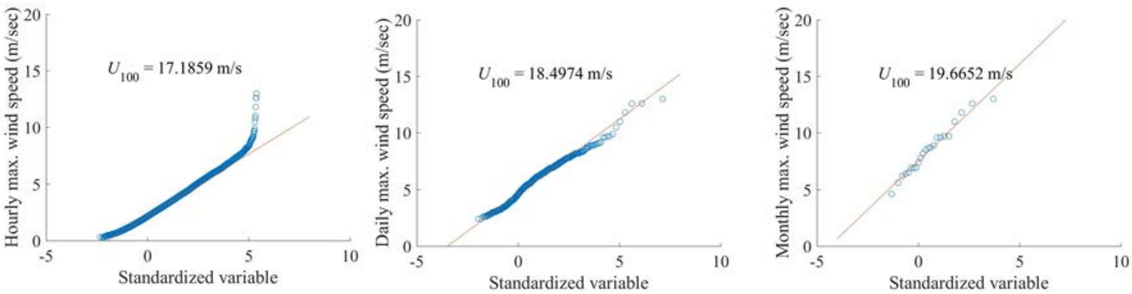
Statistical analysis of wind speed record
(at Yangon Int. Airport, May, 2014 to April, 2016)



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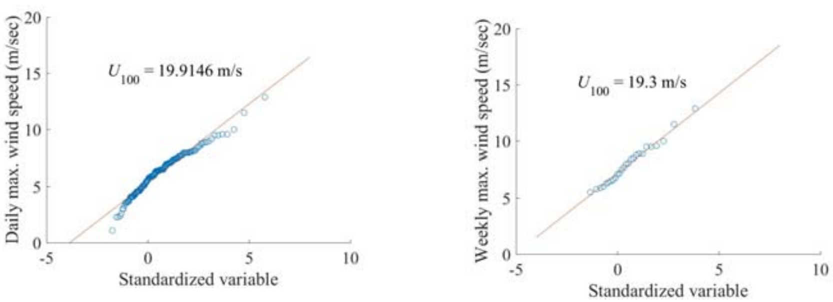
Statistical analysis of wind speed record
(at Power plant in Thilawa, August, 2015 to May, 2016)



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Recommendation to Cable vibration

The cable of the bridge may suffer from aerodynamic vibration such as rain wind induced vibration. According to the Wind resistant design handbook [1], cables are estimated to be stable for wind if their Scruton number Sc ($Sc=2m\delta/\rho D^2$, m: mass of a cable per unit length, δ: logarithmic damping decrement, ρ: air density [kg/m^3] and D: cable diameter [m]) is 140 to 200, which corresponds to d = about 0.02 to 0.03.

Cable vibration is one of the academic research subjects and there is still discussion on its generation mechanism and the effect of wind turbulence. Therefore, it should be conservative to recommend to install a rubber damper in order to achieve the above damping.

To install pedestals on the deck is also recommended. This enables to install a conventional damper if cable vibration will become a major concern after completion.

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Wind Resistance on Structures (Wind Engineering)

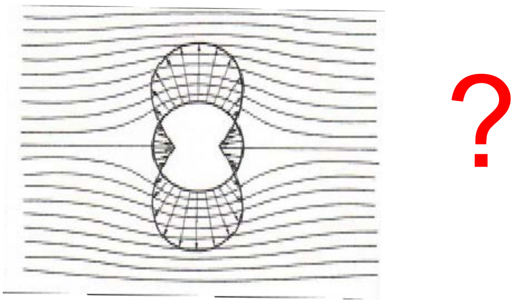
Hiromichi Shirato
Professor, Bridge Engineering Lab.
Dep't. of Civil and Earth Resources Eng.

1) Aerodynamics of structures (Bluff body aerodynamics)
17 and 19, Jan., 2018
2) Characteristics of natural wind and Design wind load
23, Jan., 2018
3) Wind resistant design
26, Jan., 2018

1) Aerodynamics of structures

-Bluff body aerodynamics-

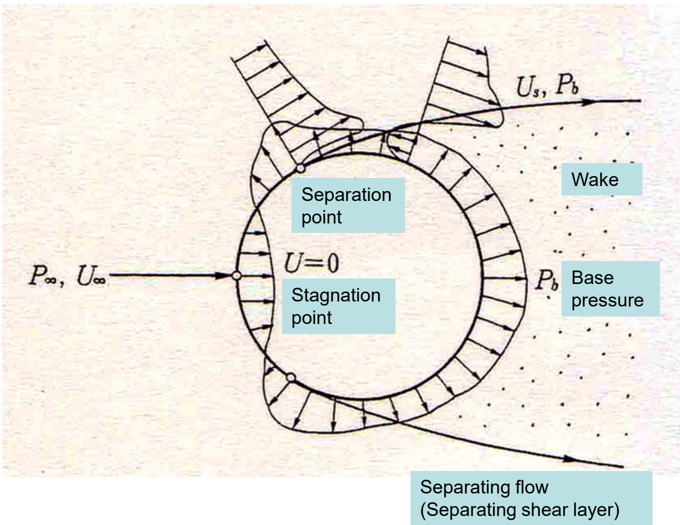
Flow around a circular cylinder in perfect fluid ($Re \rightarrow \infty$)



D'Alembert's paradox

3

Pressure distribution on surface of a circular cylinder in real flow



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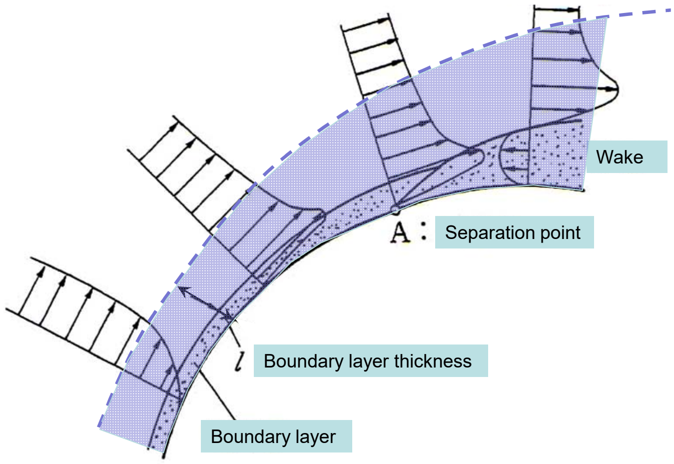
Boundary layer on surface of a circular cylinder and flow separation



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Turbulence transition near the critical Reynolds number

Subcritical

(Re=260～2.8×10^5)

Critical

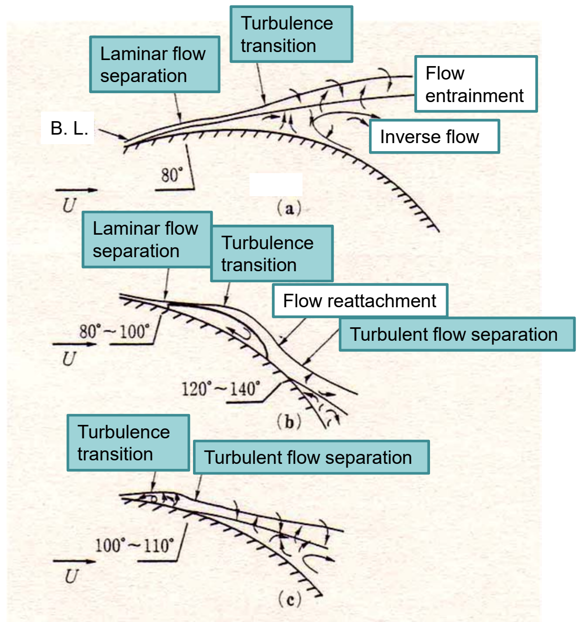
(Re=2.8～3.5×10^5)

Supercritical

(Re=3.5～15×10^5)

Hypercritical

(Re>1×10^6)



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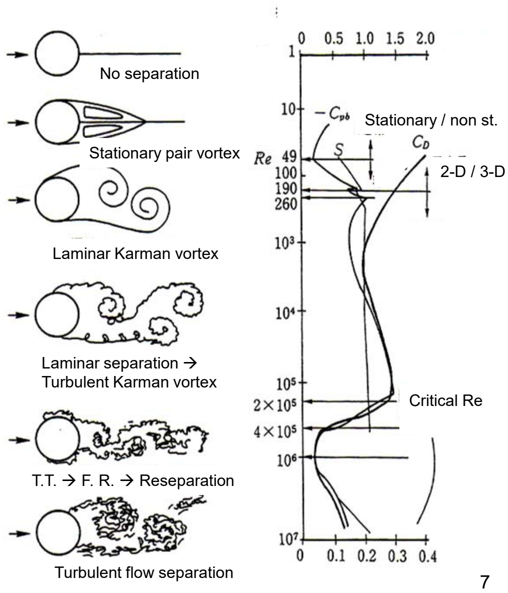
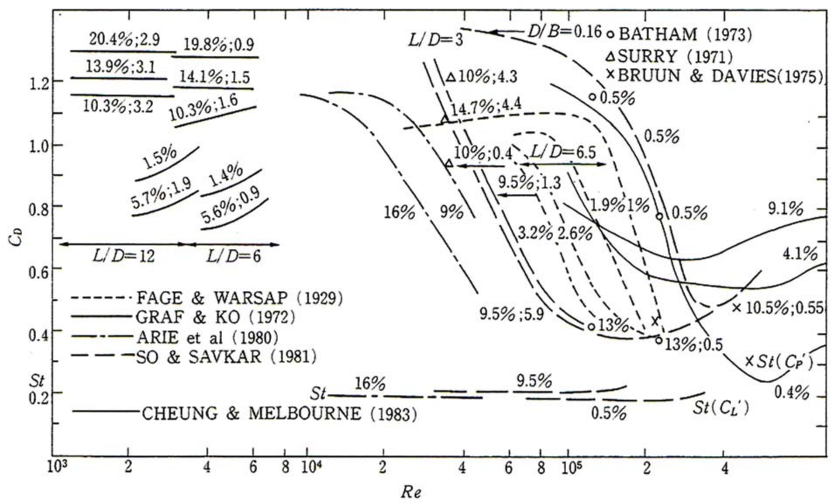
Flow pattern around a circular cylinder and its aerodynamic characteristics with Reynolds number



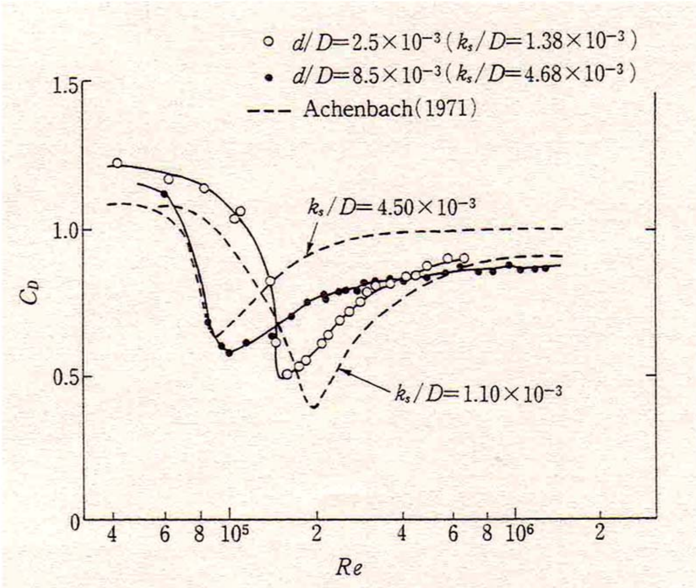
7

Effect of turbulence on C_D of a circular cylinder



8

Effect of surface roughness on C_D of a circular cylinder



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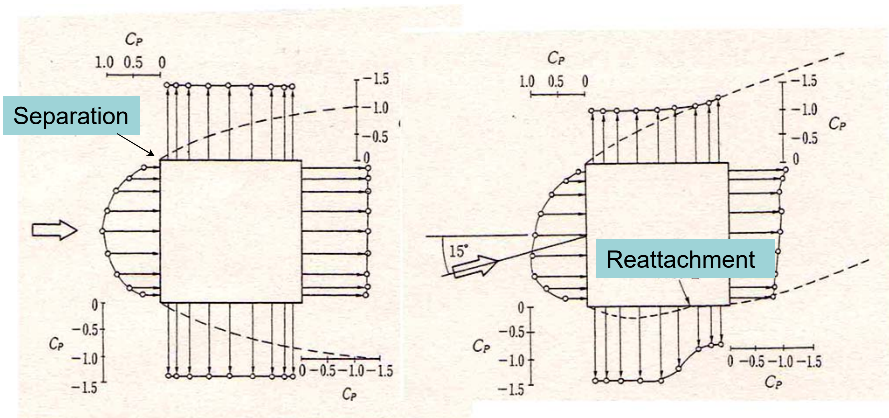
Surface pressure distribution and time integrated flow pattern around a rectangular prism (b/d=1.0)



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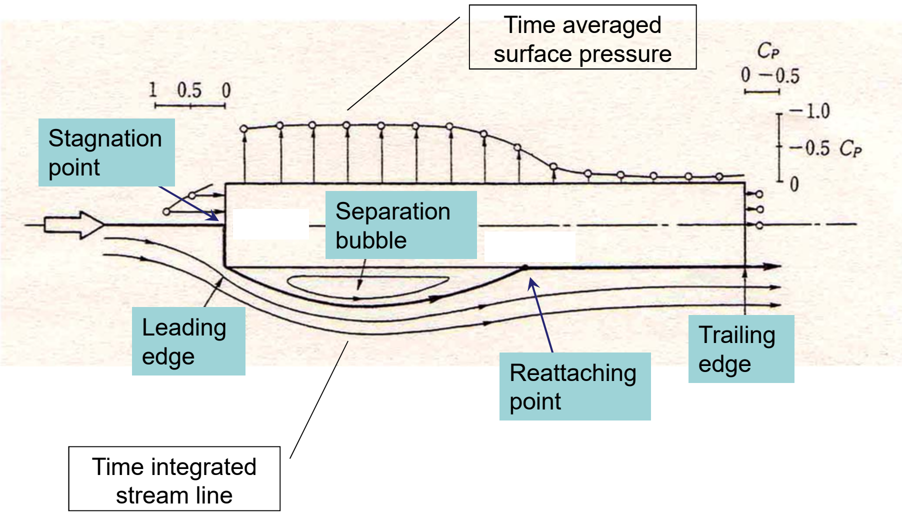
Surface pressure distribution and time integrated flow pattern around a rectangular prism (b/d>2.8)



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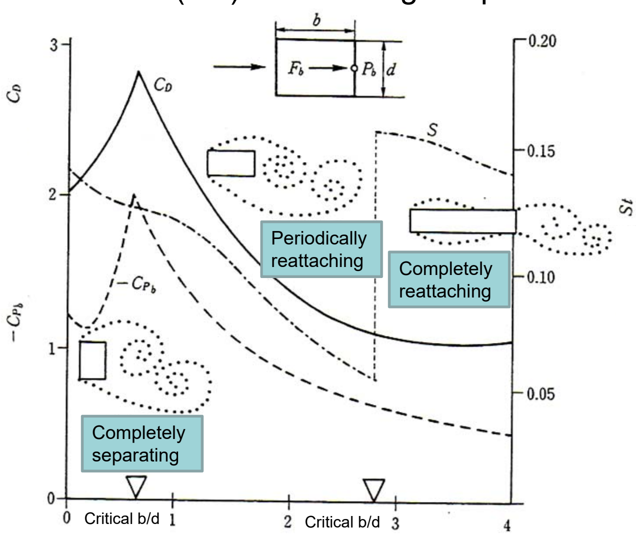
Dependence of aerodynamic characteristics

on slenderness ratio (b/d) of a rectangular prism



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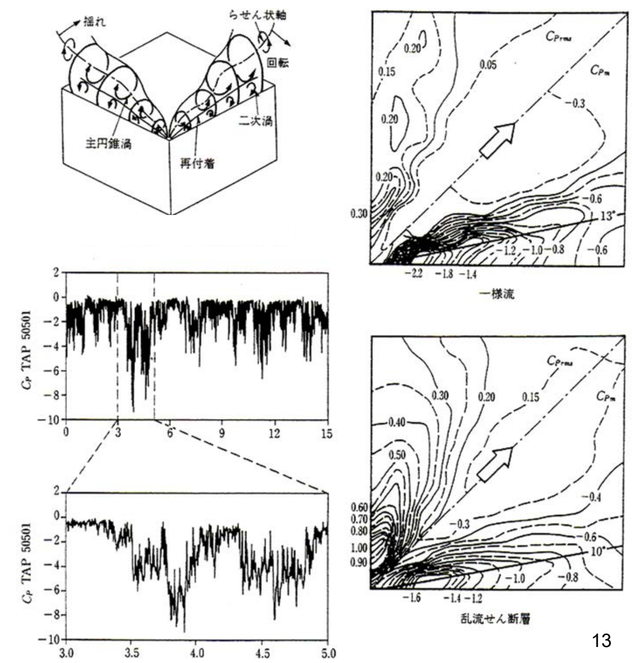
Conical vortex and surface pressure on roof of a flat-roof building



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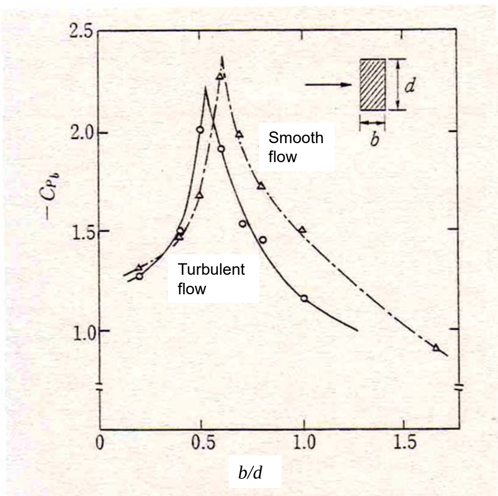
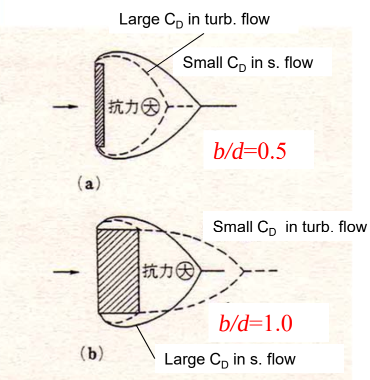
Effect of turbulence on C_D of a rectangular prism





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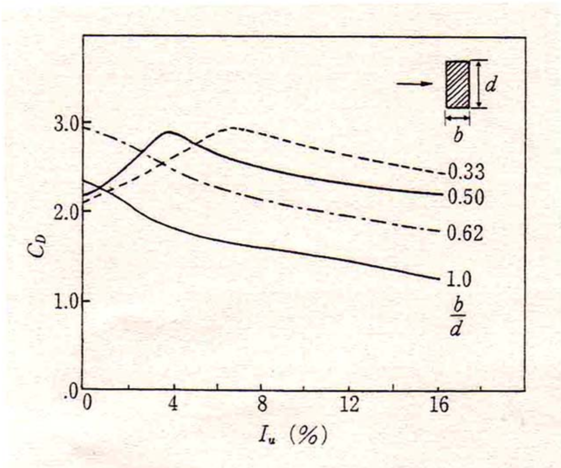
Dependence of C_D on turbulence intensity



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Aerodynamic phenomena

- Vortex-induced vibration
- Galloping
- Flutter
- Gust response, Buffeting
- Cable vibration

  Wake galloping

  Rain-wind-induced vibration

  Dry state galloping

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- Vortex-Induced Vibration, VIV

An aerodynamic vibration phenomenon due to **synchronization** of Karman vortex shedding frequency and natural frequency of structure (Karman vortex type VIV, KV)

An aerodynamic vibration phenomenon due to the **separation vortex** at the leading edge which is generated by the motion of structure itself and its growth up by amalgamating with another separation vortex at the trailing edge (Movement-induced type VIV, MIV)

VIV appears as an harmonic vibration with limited amplitude under constant wind velocity

VIV occurs in heaving mode (transverse, cross wind direction to approaching wind), in torsional mode (twisting mode around the member axis of structure) and in in-line mode (longitudinalmode)

VIV occurs only in limited wind velocity region

The formula to predict the onset wind velocity of VIV is proposed.

KV $U_{KVh} = \frac{1}{St} \cdot f_h \cdot D$ (for heaving), $U_{KVt} = \frac{1}{St} \cdot f_t \cdot D$ (for torsional)

MIV $U_{MIVh} = 1.67 \cdot B \cdot f_h$ (for heaving), $U_{MIVt} = 1.11 \cdot B \cdot f_t$ (for torsional)

St : Strouhal number
B : Width of cross section [m]
D : Depth of cross section [m]
f_h : Natural frequency of heaving mode [Hz]
f_t Natural frequency of torsional mode [Hz]

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- Galloping

An aerodynamic vibration phenomenon which occurs when the direction of the quasi-steady lift force(*) corresponding to the relative incidence angle of wind(**) is identical to the direction of heaving vibration.

* Time averaged lift force for a still cross section for the relative incidence angle of wind

** The vertical angle of the relative wind velocity being composed by the horizontal approaching wind U and the velocity of heaving displacement dy/dt

Galloping appears as an harmonic vibration with diverging heaving amplitude .

Galloping occurs if the approaching wind velocity is beyond the critical wind velocity.

Analytical formula to predict the critical wind velocity of Galloping is proposed. (Den Hartog's criteria)

The system is unstable for Galloping, if $\frac{dC_L}{d\alpha} < 0$.

C_L : lift force coefficient with respect to structural axis

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- Flutter

An aerodynamic vibration phenomenon induced by the time-dependent aerodynamic forces generated by self-induced motion of structure (seld-induced aerodynamic forces).

Self-induced aerodynamic forces can be measured in wind tunnel or be evaluated by the help of CFD for general cross section geometry of structures.

Self-induced aerodynamic lift and pitching moment can be obtained analytically for a thin flat plate and a thin airfoil.

Flutter appears as an harmonic vibration with diverging amplitude with torsional 1dof (torsional flutter) or with heaving and torsional 2dof (coupled flutter).

Flutter occurs if the approaching wind velocity is beyond the critical wind velocity.

Flutter analysis should be conducted to predict the critical wind velocity.

19

- Buffeting, Gust Response

Buffeting or gust response is random vibration due to wind turbulence.

Wind tunnel experiment should be conducted to evaluate aerodynamic gust forces with enough precision.

Buffeting response analysis proposed by Davenport in 1961 is applied to evaluate not only buffeting response but also to determine wind load for wind resistant design.

Buffeting can be appeared in any wind velocity, but its amplitude is proportional to the square of wind velocity.

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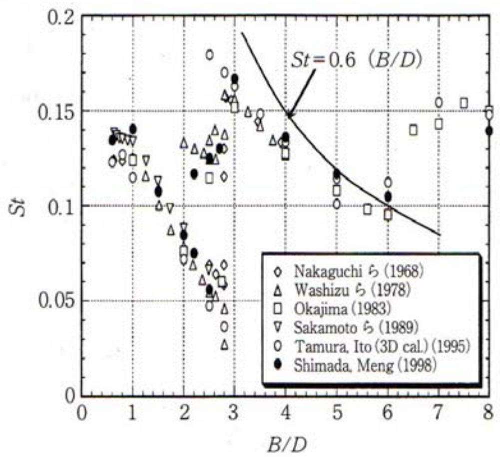
Vortex-Induced Vibration, Galloping, Flutter

Karman VIV, KV



Strouhal number of a rectangular prism

Strouhal Number

$$St = \frac{f_s D}{U}$$

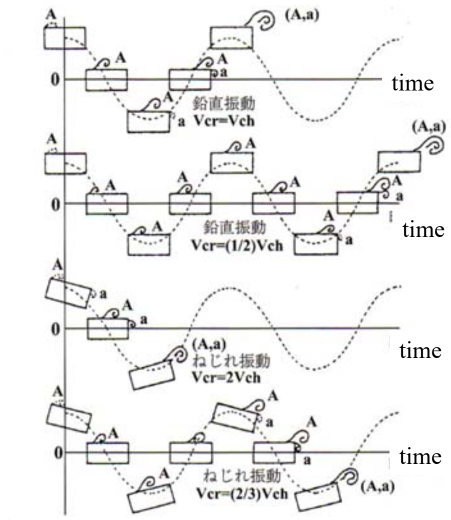
f_s : vortex shedding frequency [Hz]

D : depth of cross section [m]

U : wind velocity [m/s]

KV occurs at U=Ucr where f_s is equal to natural freq.

VIV (movement-induced type), MIV



heaving $V_{onset} = V_{ch}$

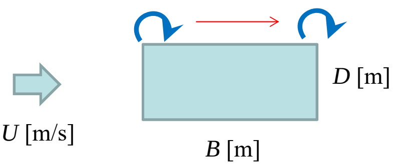
heaving $V_{onset} = \frac{1}{2} V_{ch}$

torsional $V_{onset} = 2V_{ch}$

torsional $V_{onset} = \frac{2}{3} V_{ch}$

Time for one separation vortex travelling along the side surface of cross section and reach to the trailing edge, t_{travel}

$$t_{travel} = \frac{B}{0.6U} = \begin{cases} T_h = \frac{1}{f_h} \\ 2T_h = \frac{2}{f_h} \\ 0.5T_t = \frac{1}{2f_t} \\ 1.5T_t = \frac{3}{2f_t} \end{cases}$$

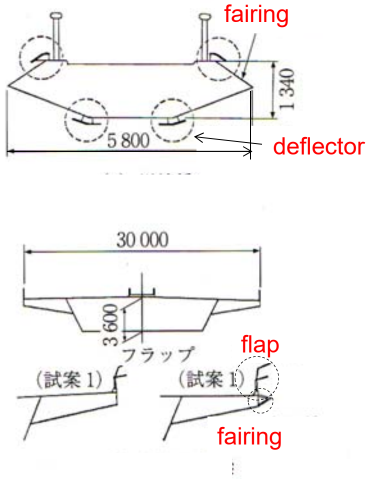


f_h : natural frequency for heaving mode [Hz]

f_t : natural frequency for torsional mode [Hz]

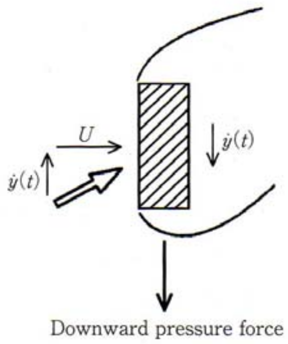
$$V_{ch} = \frac{U}{f_h D} = \frac{1}{0.6} \frac{B}{D} \cong 1.67 \frac{B}{D}$$

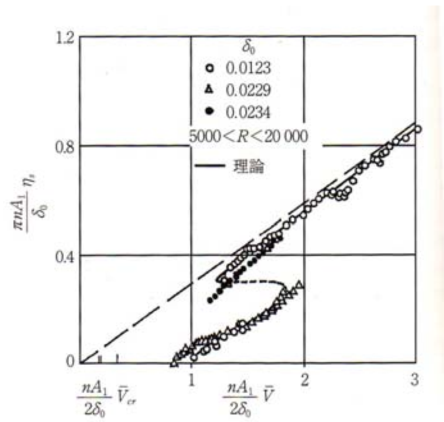
Aerodynamic countermeasure



Galloping, Flutter

Galloping





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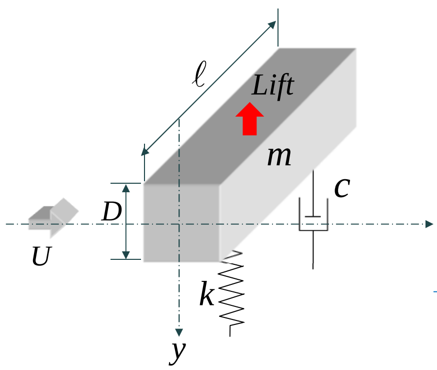
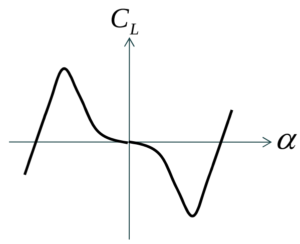
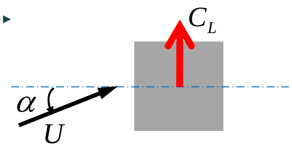
$$m\{\ddot{y}+2\varsigma\omega\dot{y}+\omega^2 y\}=-\frac{1}{2}\rho U^2 D\ell C_L(\alpha)$$

$$C_L=\frac{Lift}{\frac{1}{2}\rho U^2 D\ell}$$







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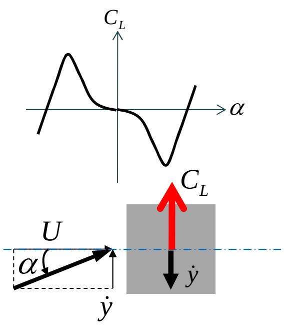
$$C_L(\alpha) = C_L(0) + \frac{dC_L}{d\alpha}\alpha - \frac{d^3C_L}{d\alpha^3}\alpha^3 + \cdots$$

$$\alpha = \tan^{-1}\left(\frac{\dot{y}}{U}\right) \cong \frac{\dot{y}}{U}, \quad C_L(0) = 0$$

$$\therefore \quad C_L\left(\frac{\dot{y}}{U}\right) = \frac{dC_L}{d\alpha}\frac{\dot{y}}{U}$$



$$m\{\ddot{y} + 2\varsigma\omega\dot{y} + \omega^2 y\} = -\frac{1}{2}\rho U^2 D\ell \frac{dC_L}{d\alpha}\frac{\dot{y}}{U}$$

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Linear equation of motion

$$m\{\ddot{y} + 2\varsigma\omega\dot{y} + \omega^2 y\} = -\frac{1}{2}\rho U^2 D\ell \frac{dC_L}{d\alpha}\frac{\dot{y}}{U}$$

damping term

$$2\omega\left(\varsigma + \frac{\rho U D\ell}{4m\omega}\frac{dC_L}{d\alpha}\right)\dot{y}$$

If $\frac{dC_L}{d\alpha} < 0$, the system will be unstable when

$$U > \frac{4m\varsigma\omega}{\rho D\ell |a_1|}$$

Den Hartog's criteria

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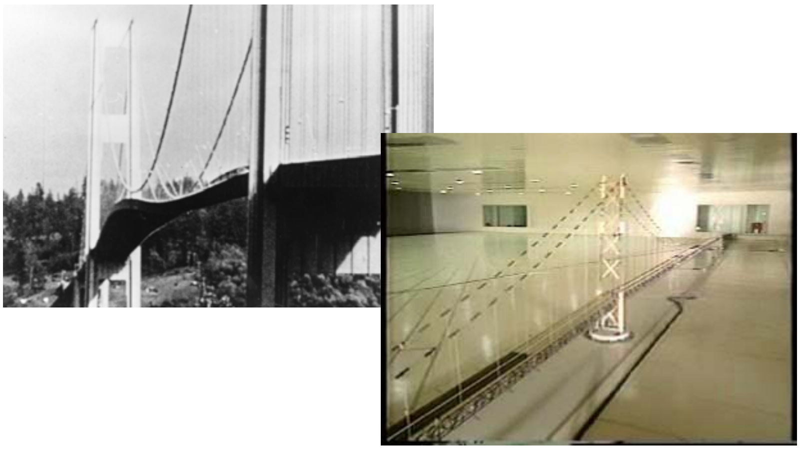
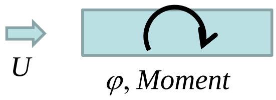
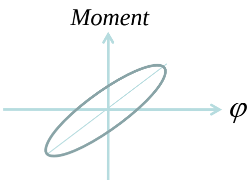
Flutter --- self-induced vibration due to motion-induced aerodynamic forces



Motion-induced Aerodynamic Force





$$\varphi = \varphi_0 \cos\omega t$$

$$\begin{aligned} M &= M_0 \cos(\omega t - \theta) \\ &= M_0 \cos\omega t \cos\theta + \sin\omega t \sin\theta \\ &= M_0 \left(\frac{\varphi}{\varphi_0} \cos\theta - \frac{\dot{\varphi}}{\varphi_0 \omega} \sin\theta \right) \end{aligned}$$

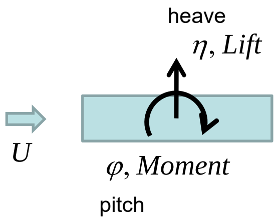
$$\begin{aligned} W &= \oint M d\varphi \\ &= \int_0^{2\pi/\omega} M \dot{\varphi}\, dt \\ &= -\pi M_0 \varphi_0 \sin\theta \end{aligned}$$

negative workdone for positive θ

Linear system of 2 dof

$$L = \frac{1}{2}\rho(2b)U^2\left(kH_1^*\frac{\dot{\eta}}{U} + kH_2^*\frac{b\dot{\varphi}}{U} + k^2H_3^*\varphi + k^2H_4^*\frac{\eta}{b}\right)$$

$$M = \frac{1}{2}\rho(2b^2)U^2\left(kA_1^*\frac{\dot{\eta}}{U} + kA_2^*\frac{b\dot{\varphi}}{U} + k^2A_3^*\varphi + k^2A_4^*\frac{\eta}{b}\right)$$



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$$[M]\{\ddot{Z}\} + [C]\{\dot{Z}\} + [K]\{Z\} = [A]\{\dot{Z}\} + [B]\{Z\}$$

$$\{Z\} = \begin{Bmatrix}\eta_0 \\ \varphi_0\end{Bmatrix} e^{\lambda t} = \{Z_0\}e^{\lambda t}$$

$$[M] = \begin{bmatrix} m & 0 \\ 0 & I \end{bmatrix},\ [C] = \begin{bmatrix} C_\eta & 0 \\ 0 & C_\varphi \end{bmatrix} = \begin{bmatrix} 2m\varsigma_\eta\omega_\eta & 0 \\ 0 & 2I\varsigma_\varphi\omega_\varphi \end{bmatrix},$$

$$[K] = \begin{bmatrix} k_\eta & 0 \\ 0 & k_\varphi \end{bmatrix} = \begin{bmatrix} m\omega_\eta^2 & 0 \\ 0 & I\omega_\varphi^2 \end{bmatrix}$$

$$[A] = \begin{bmatrix} \rho b^2\omega H_1^* & \rho b^3\omega H_2^* \\ \rho b^3\omega A_1^* & \rho b^4\omega A_2^* \end{bmatrix},\ [B] = \begin{bmatrix} \rho b^2\omega^2 H_4^* & \rho b^3\omega^2 H_3^* \\ \rho b^3\omega^2 A_4^* & \rho b^4\omega^2 A_3^* \end{bmatrix}$$

State vector $\{Y\} = \begin{Bmatrix} \lambda[Z] \\ [Z] \end{Bmatrix}$

$[A^*]\{Y\} = \lambda[\mu]\{Y\}$ Eigen value problem

$$[A^*] = \begin{bmatrix} -([C]-[A]) & -([K]-[B]) \\ [M] & [0] \end{bmatrix},\ [\mu] = \begin{bmatrix} [M] & [0] \\ [0] & [M] \end{bmatrix}$$

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Eigen equation

$$\left|\left[A^*\right]-\lambda\left[\mu\right]\right|=0$$

Eigen value

$$\lambda_j=\left(-\varsigma_j+i\right)\omega_j \qquad \left(j=1,2\right)$$

ς_j : critical damping ratio for *j*-th mode

ω_j : natural circular frequency for *j*-th mode

$$\{Z\}=\begin{Bmatrix}\eta_0\\ \varphi_0\end{Bmatrix}e^{\lambda t}=\{Z_0\}e^{\lambda t}$$

Amplitude vector for *j*-th mode

$$\{Z_{0j}\}=\begin{Bmatrix}\eta_{0j}\\ \varphi_{0j}\end{Bmatrix}$$

Amplitude ratio, phase for *j*-th mode

$$R_j=\frac{\left|\eta_{0j}\right|}{\left|\varphi_{0j}\right|}, \qquad \Psi_j=\tan^{-1}\frac{C_{Ij}}{C_{Rj}} \qquad \left(C_{Rj}+iC_{Ij}=\frac{\eta_{0j}}{\varphi_{0j}}\right)$$

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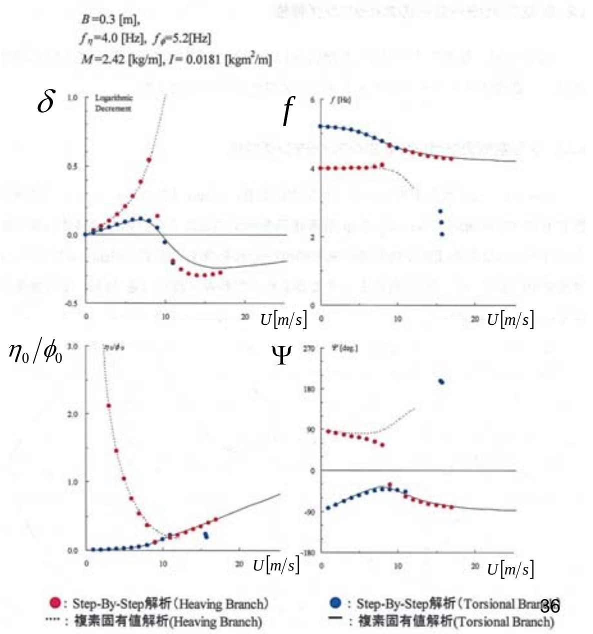
Flutter analysis



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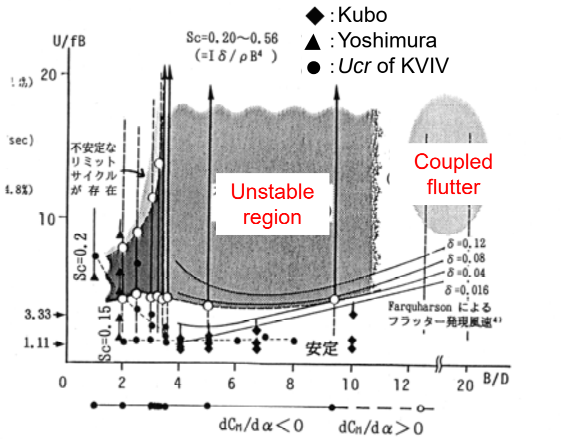
Torsional flutter



Aerodynamically unstable region for torsional flutter on a 2-D H-shaped section model (in smooth flow, for $2A_t$=1 [deg])

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Buffeting

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Buffeting (Gust Response)

- Structural response due to sudden change of wind velocity ("gust", "gusty wind")
- Structural response due to turbulence
- Random vibration
- Maximum wind vel., turbulence, structural dynamics, max. displ./stress
- Important from design point of view

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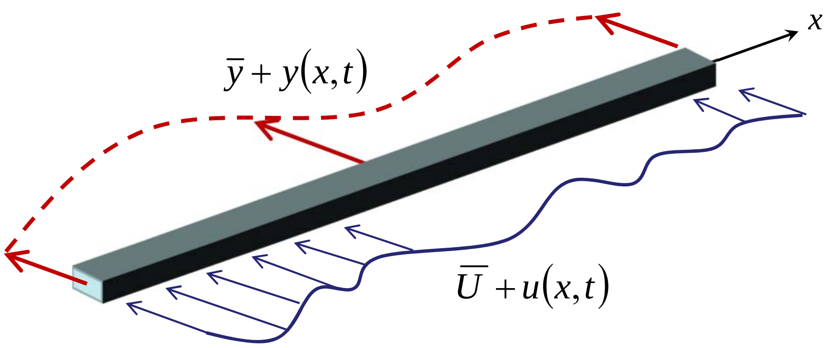
Equation of motion for along wind displacement of a girder $y(x,t)$

$$m(x)\ddot{y}(x,t)+c(x)\dot{y}(x,t)+\left(EI(x)y''\right)''=p(x,t)$$



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Power spectral density (PSD) of along wind displacement $y(x,t)$ of a bridge girder element at location x along bridge axis $S_y(x; f)$

$$S_y(x; f) = \sum_j S_{q_{y_j}}(f)\phi_j^2(x)$$

$$S_{q_{y_j}}(f) = \frac{4\overline{P}^2\left\{\int_L \phi_j^2(x)dx\right\}^2}{\overline{U}^2\omega_j^4\left\{\int_L m(x)\phi_j^2(x)dx\right\}^2}\left|\chi_{d_j}(f)\right|^2\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)$$

$S_{q_{y_j}}(f)$: PSD of generalized coordinate for j-th mode of $q_{y_j}(t)$

$\phi_j(x)$: j-th modal function (mode vector)

$\overline{P} = \frac{1}{2}\rho\overline{U}^2C_D(0)D$ $\overline{P}$: time averaged drag force per unit span length (drag force due to average wind speed)

ρ : air density $[\mathrm{kg/m^3}]$ $\overline{U}$: average wind speed $[\mathrm{m/s}]$

$C_D(0)$: drag force coefficient D : girder height $[\mathrm{m}]$

PSD: one-sided spectrum

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$\left|\chi_{d_j}(f)\right|^2$: dynamic magnification for j-th mode

$\left|\chi_A(f)\right|^2$: aerodynamic admittance

$\left|J_j(f)\right|^2$: joint mode acceptance

$S_u(f)$: PSD of wind speed fluctuation $u(t)$ $[\mathrm{m^2/s^2/Hz}]$

PSD: one-sided spectrum

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Expected maximum value for displacement and sectional force

displacement: $S_y(x;f)=\sum_j S_{q_{y_j}}(f)\phi_j^{\,2}(x)$

bending moment: $S_M(x;f)=\{EI(x)\}^2\sum_j S_{q_{y_j}}(f)\{\phi''(x)\}^2$

shear force: $S_Q(x;f)=\sum_j S_{q_{y_j}}(f)[\{EI(x)\phi''(x)\}']^2$

$$\bar{y}_{\max}=G\bar{y}=\bar{y}\left\{1+g(\nu T)\frac{\sigma_x}{\bar{y}}\right\}$$

G: gust factor

$$g(\nu T)=\sqrt{2\ln\nu T}+\frac{\alpha}{\sqrt{2\ln\nu T}}$$

$g(\nu T)$: peak gust factor

α=1, 0.5772

$$\nu=\left\{\frac{\int_0^\infty f^2 S_y(f)df}{\int_0^\infty S_y(f)df}\right\}^{\frac{1}{2}}$$

ν : peak response factor

$T=600[s]$

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Cable vibration

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Design of Cable-Stayed Bridge

- Stay-cables are vibrating on many bridges in all over the world.
- Mechanisms of wind-induced instabilities have not totally clarified yet.
- Mitigation of wind-induced cable vibration is still one of the most important issue on the design of cable-stayed bridges.

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Rain-Wind Induced Vibration

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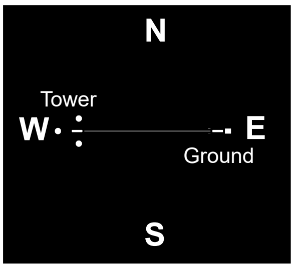
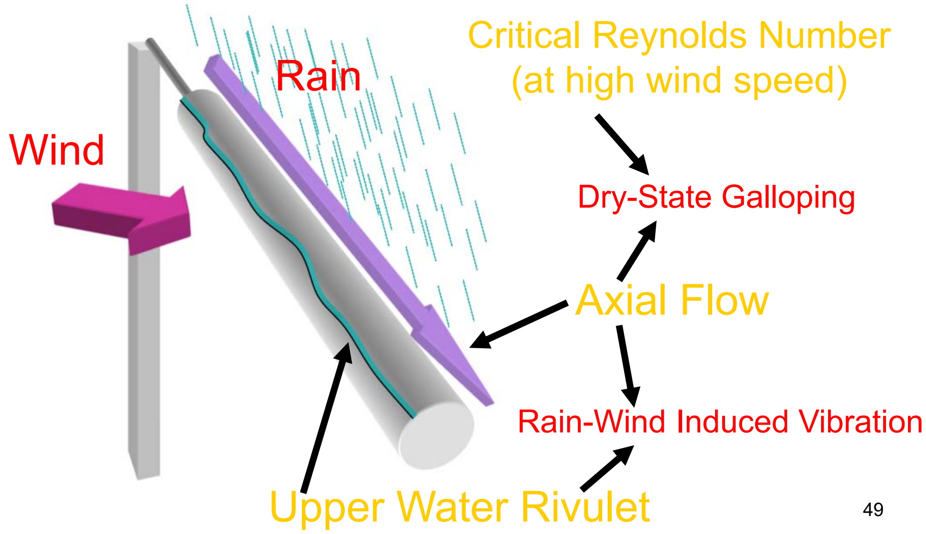
Galloping??

August 21, 2001 12:21 - 12:41
Typhoon Pabuk (T0111)

Mean Wind Velocity:
13.2m/s (20 min.)
Mean Wind Direction:
126 deg. (ESE-SE)
Maximum Wind Speed:
35.3m/s
Rainfall:
37mm/h



Generation Mechanisms of Wind-Induced Cable Vibration



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Cable Aerodynamics of Single Cable

- Vortex-induced vibration (with/without rain)
  - Strouhal number: $St=f_sD/U$=0.2 (circular cylinder)
  - $St=f_sD/U$=0.1-0.2 (inclined circular cylinder)
  - Onset wind velocity: U/fD=5 (circular cylinder)
  - U/fD=5-10 (inclined circular cylinder)
- Rain-wind induced vibration (with rain)
  - Water rivulet
  - Axial flow
  - etc.
- Dry-state galloping (without rain)
  - Axial flow
  - Critical Reynolds number
  - etc.

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Cable Aerodynamics of Parallel Cables

- Aerodynamics of single cable
- Wake galloping
  - Cable separation < 5 or 6D
  - Onset wind velocity *U/fD*>20
  - 1st mode vibration
  - Sub-span vibrations (with spacers)
- Wake-induced flutter
  - Cable separation = 8 or 10 - 20D
  - Parallel hanger ropes of Akashi Kaikyo Bridge

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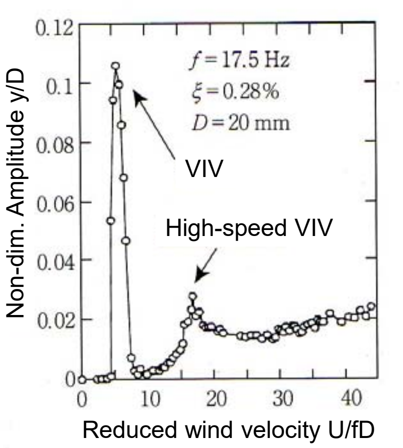
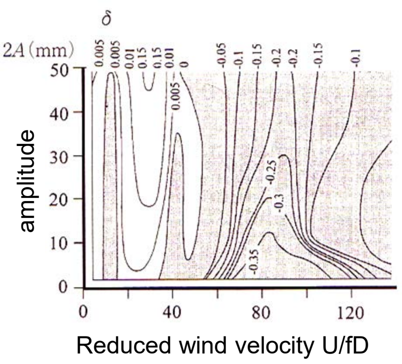
High-speed VIV



VIV for cylindrical tower



VIV, Galloping and High-speed VIV for an inclined circular cylinder

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Symmetrical vortex

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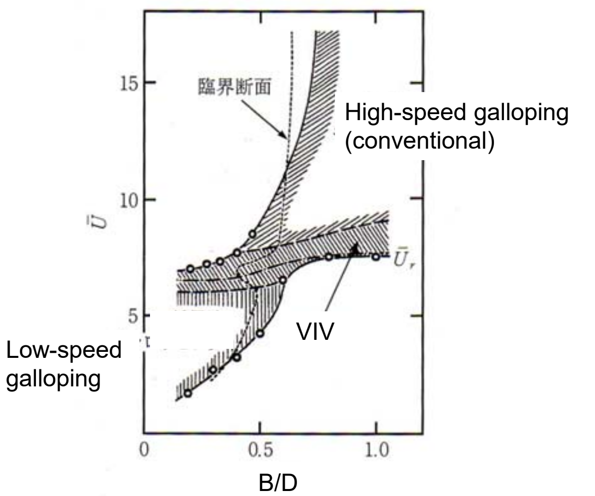
High-speed galloping and low-speed galloping



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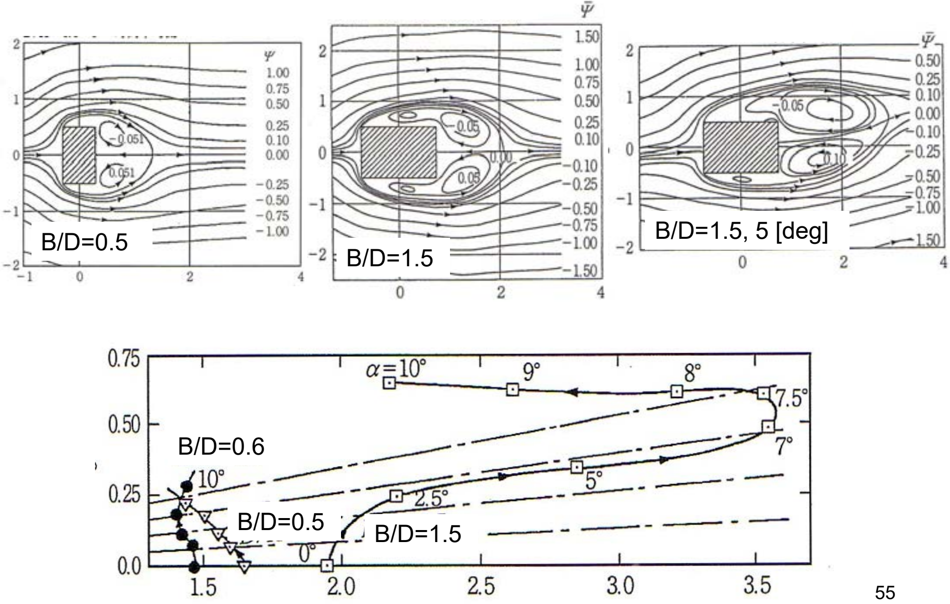
Time integrated streamline and wake stagnation point



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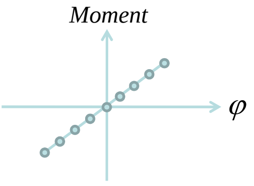
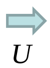
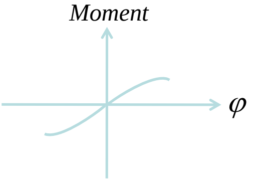
Motion-induced Aerodynamic Force



$\varphi = \varphi_0 \cos \omega t$

$M = M_0 \cos \omega t$





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Linear system of multi-degree of freedom
--Flutter equation--

$$m\left(\ddot{\eta}+2\varsigma_{\eta 0}\omega_{\eta 0}\dot{\eta}+\omega_{\eta 0}{}^{2}\eta\right)=\frac{1}{4}\rho U^{2}B\left(kH_1{}^{*}\frac{\dot{\eta}}{U}+kH_2{}^{*}\frac{B\dot{\varphi}}{U}+k^{2}H_3{}^{*}\varphi+k^{2}H_4{}^{*}\frac{\eta}{U}+kH_5{}^{*}\frac{\dot{p}}{U}+k^{2}H_6{}^{*}\frac{p}{B}\right)$$

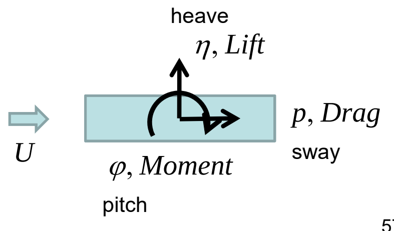
$$m\left(\ddot{p}+2\varsigma_{p0}\omega_{p0}\dot{p}+\omega_{p0}{}^{2}p\right)=\frac{1}{4}\rho U^{2}B\left(kP_1{}^{*}\frac{\dot{\eta}}{U}+kP_2{}^{*}\frac{B\dot{\varphi}}{U}+k^{2}P_3{}^{*}\varphi+k^{2}P_4{}^{*}\frac{\eta}{U}+kP_5{}^{*}\frac{\dot{p}}{U}+k^{2}P_6{}^{*}\frac{p}{B}\right)$$

$$I\left(\ddot{\varphi}+2\varsigma_{\varphi 0}\omega_{\varphi 0}\dot{\varphi}+\omega_{\varphi 0}{}^{2}\varphi\right)=\frac{1}{4}\rho U^{2}B^{2}\left(kA_1{}^{*}\frac{\dot{\eta}}{U}+kA_2{}^{*}\frac{B\dot{\varphi}}{U}+k^{2}A_3{}^{*}\varphi+k^{2}A_4{}^{*}\frac{\eta}{U}+kA_5{}^{*}\frac{\dot{p}}{U}+k^{2}A_6{}^{*}\frac{p}{B}\right)$$

The aerodynamic derivatives,

$$H_i^{*}, P_i^{*}, A_i^{*}\,(i=1,2,3,4,5,6)$$

are functions of $k\left(=\frac{b\omega}{U}\right)$.



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Spectral density

Frequency distribution of $x(t)$

A sample function is generally,

1) non-periodic → Fourier analysis cannot be applied.
2) infinite data → The condition of $\int_{-\infty}^{\infty}|x(t)|dt<\infty$ is not satisfied, and therefore Fouries transform cannot be applied.

⇓

Auto correlation function $R_x(\tau)$, instead of $x(t)$

$x(t)$ and $x(t+\tau)$ are in phase : $R_x(\tau)=1$ (fully correlated)

$x(t)$ and $x(t+\tau)$ are out of phase : $R_x(\tau)=-1$ (fully anti-correlated)

→ Frequency dist. of $R_x(\tau)$ represents those of $x(t)$.

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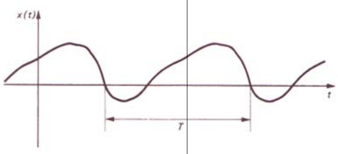
Fourier analysis

Arbitrary periodic function

$$x(t) = a_0 + a_1\cos\frac{2\pi t}{T} + a_2\cos\frac{4\pi t}{T} + \cdots$$
$$+ b_1\sin\frac{2\pi t}{T} + b_2\sin\frac{4\pi t}{T} + \cdots$$
$$= a_0 + \sum_{k=1}^{\infty}\left(a_k\cos\frac{2\pi kt}{T} + b_k\sin\frac{2\pi kt}{T}\right)$$



$$a_0 = \frac{1}{T}\int_{-T/2}^{T/2} x(t)dt$$
$$a_k = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\cos\frac{2\pi kt}{T}dt$$
$$b_k = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\sin\frac{2\pi kt}{T}dt$$

$\omega_k = \frac{2\pi k}{T}$ *k*-th frequency

$\Delta\omega = \frac{2\pi}{T}$ Frequency interval

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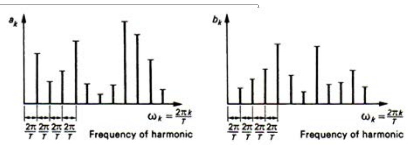
Fourier integral

Non-periodic function $(T \to \infty)$

$$a_0 = 0$$
$$a_k = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\cos\frac{2\pi kt}{T}dt = \frac{\Delta\omega}{\pi}\int_{-T/2}^{T/2} x(t)\cos\omega_k t dt$$
$$b_k = \frac{2}{T}\int_{-T/2}^{T/2} x(t)\sin\frac{2\pi kt}{T}dt = \frac{\Delta\omega}{\pi}\int_{-T/2}^{T/2} x(t)\sin\omega_k t dt$$



$$\to \begin{cases} \alpha(\omega)d\omega = \frac{d\omega}{\pi}\int_{-\infty}^{\infty} x(t)\cos\omega t dt \\ \beta(\omega)d\omega = \frac{d\omega}{\pi}\int_{-\infty}^{\infty} x(t)\sin\omega t dt \end{cases} \quad (\omega \ge 0)$$

$$\begin{cases} A(\omega)d\omega = \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)\cos\omega t dt \\ B(\omega)d\omega = \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)\sin\omega t dt \end{cases} \quad (-\infty < \omega < \infty)$$

$$x(t) = 2\int_0^{\infty} A(\omega)\cos\omega t d\omega + 2\int_0^{\infty} B(\omega)\sin\omega t d\omega$$

Necessary and sufficient condition for the integral to be finite

$$\int_{-\infty}^{\infty} |x(t)| dt < \infty$$

$x(t)$ should approaches to 0 at $t \to |\infty|$

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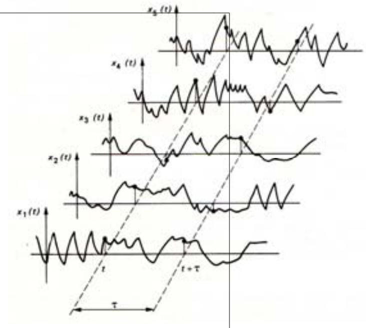
Autocorrelation function

$$R_x(\tau) = \lim_{t \to \infty} \frac{1}{T} \int_0^T x(t)\,x(t+\tau)\,dt$$

random process $\rightarrow$ function of τ (not t)

$$E[x(t)x(t+\tau)] = f(\tau) \equiv R_x(\tau)$$

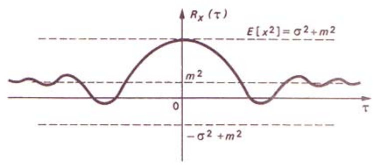
$$\rho_{x(t)x(t+\tau)} = \frac{E[\{x(t)-m\}\{x(t+\tau)-m\}]}{\sigma_x^{\,2}} = \frac{R_x(\tau)-m^2}{\sigma_x^{\,2}}$$



$$\therefore \quad R_x(\tau) = \sigma_x^{\,2}\rho + m^2$$

$$-\sigma_x^{\,2} + m^2 \;\le\; R_x(\tau) \;\le\; \sigma_x^{\,2} + m^2 \;\left(= E[x^2]\right)$$

$$R_x(\infty) \rightarrow m^2$$



Cross-correlation function

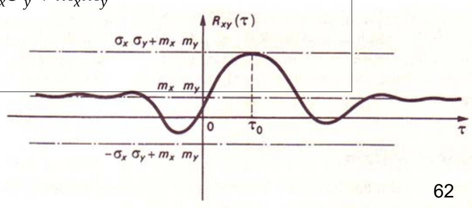
$$R_{xy}(\tau) = E[x(t)y(t+\tau)] \qquad R_{yx}(\tau) = E[y(t)x(t+\tau)]$$

- $R_{xy}(-\tau) = E[x(t)y(t-\tau)] = R_{yx}(\tau)$
- $R_{yx}(-\tau) = E[y(t)x(t-\tau)] = R_{xy}(\tau)$

- $R_{xy}(\tau) = \sigma_x\sigma_y\rho_{xy}(\tau) + m_x m_y$
- $R_{yx}(\tau) = \sigma_y\sigma_x\rho_{yx}(\tau) + m_y m_x$

- $-\sigma_x\sigma_y + m_x m_y \;\le\; R_{xy}(\tau) \;\le\; \sigma_x\sigma_y + m_x m_y$

- $R_{xy}(\infty) \rightarrow m_x m_y$
- $R_{yx}(\infty) \rightarrow m_y m_x$



Fourier transform

$$e^{i\theta} = \cos\theta + i\sin\theta$$

$$X(\omega) \equiv A(\omega) - iB(\omega)$$
$$= \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)(\cos\omega t - i\sin\omega t)dt$$
$$= \frac{1}{2\pi}\int_{-\infty}^{\infty} x(t)e^{-i\omega t}dt \quad \leftarrow \text{ Fourier transform of } x(t)$$

$$x(t) = \int_{-\infty}^{\infty} A(\omega)\cos\omega t d\omega + \int_{-\infty}^{\infty} B(\omega)\sin\omega t d\omega$$
$$+ i\int_{-\infty}^{\infty} A(\omega)\sin\omega t d\omega - \int_{-\infty}^{\infty} B(\omega)\cos\omega t d\omega$$
$$= \int_{-\infty}^{\infty} \{A(\omega) - iB(\omega)\}\{\cos\omega t + i\sin\omega t\}d\omega$$
$$= \int_{-\infty}^{\infty} X(\omega)e^{i\omega t}d\omega \quad \leftarrow \text{ Inverse Fourier transform of } X(\omega)$$

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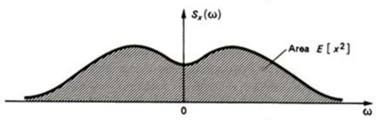
Spectral density

$$m = E[x] = 0 \quad \rightarrow \quad R_x(\infty) \rightarrow 0$$

$\therefore \int_{-\infty}^{\infty} |R_x(\tau)| d\tau < \infty$ is satisfied.



$$S_x(\omega) \equiv \frac{1}{2\pi}\int_{-\infty}^{\infty} R_x(\tau)e^{-i\omega\tau}d\tau$$ ← $S_x(\omega)$ is a spectral density of $x(t)$

$$R_x(\tau) = \int_{-\infty}^{\infty} S_x(\omega)e^{i\omega\tau}d\omega$$

$S_x(\omega)$ and $R_x(\tau)$ are pair of Fourier transform.

- $R_x(0) = \int_{-\infty}^{\infty} S_x(\omega)d\omega = E[x^2]$
- $S_x(\omega)$: real function $(\because R_x(-\tau) = R_x(\tau))$
- $S_x(\omega) = \lim_{T\to\infty} \frac{2\pi X(\omega)X^*(\omega)}{T}$

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Cross-spectral density

$$S_{xy}(\omega)=\frac{1}{2\pi}\int_{-\infty}^{\infty}R_{xy}(\tau)e^{-i\omega\tau}d\tau=C_{xy}(\omega)-iQ_{xy}(\omega)$$

$$R_{xy}(\tau)=\int_{-\infty}^{\infty}S_{xy}(\omega)e^{i\omega\tau}d\omega$$

$$\int_{-\infty}^{\infty}\left|R_{xy}(\tau)\right|d\tau<\infty \quad \leftarrow \quad m_x \quad or \quad m_y=0$$

- $S_{yx}(\omega)=\overline{S_{xy}(\omega)}=C_{xy}(\omega)+iQ_{xy}(\omega)$

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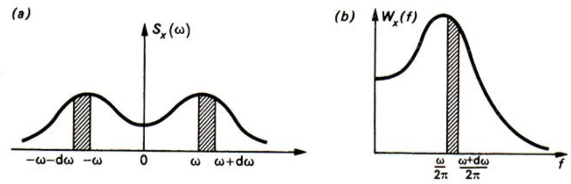
One-sided, Two-sided spectral density

$$E\left[x^2\right]=\int_{-\infty}^{\infty}S_x(\omega)d\omega$$

$$E\left[x^2\right]=\int_0^{\infty}W_x(f)df$$ Equivalent one-sided spectral density

$$2S_x(\omega)d\omega=W_x(f)df=W_x(f)\frac{d\omega}{2\pi}$$

$$\therefore \quad W_x(f)=4\pi S_x(\omega)$$



66

Equation of motion for along wind displacement of a girder $y(x,t)$

$$m(x)\ddot{y}(x,t)+c(x)\dot{y}(x,t)+\left(EI(x)y''\right)''=p(x,t)$$

$$y(x,t)=\sum_j q_{y_j}(t)\phi_j(x) \qquad \phi_j(x)\ :\ j\text{-th modal function (mode vector)}$$

$$p(x,t)=\sum_j q_{p_j}(t)\phi_j(x) \qquad \int_L \phi_i(x)\phi_j(x)dx=\delta_{ij}=\begin{cases}1 & i=j\\ 0 & i\neq j\end{cases}$$

$$\int_L m(x)\ddot{y}(x,t)\phi_j(x)dx=\int_L m(x)\sum_i \ddot{q}_{y_i}(t)\phi_i(x)\phi_j(x)dx=\int_L m(x)\phi_j^{\,2}(x)dx\cdot\ddot{q}_{y_j}(t)$$

$$\ddot{q}_{y_j}+2\varsigma_j\omega_j\dot{q}_{y_j}+\omega_j^{\,2}q_{y_j}=\frac{\int_L p(x,t)\phi_j(x)dx}{\int_L m(x)\phi_j^{\,2}(x)dx}=\frac{q_{p_j}(t)\int_L \phi_j^{\,2}(x)dx}{\int_L m(x)\phi_j^{\,2}(x)dx}$$

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Fourier transform $\hat{q}_{y_j}(\omega)=\mathcal{F}\left[q_{y_j}(\omega)\right]=\frac{1}{2\pi}\int_{-\infty}^{\infty}q_{y_j}(t)e^{-i\omega t}dt$

$$\left(-\omega^2+i2\varsigma_j\omega_j\omega+\omega_j^{\,2}\right)\hat{q}_{y_j}(\omega)=\frac{\hat{q}_{p_j}(\omega)\int_L \phi_j^{\,2}(x)dx}{\int_L m(x)\phi_j^{\,2}(x)dx}$$

$$\frac{\int_L \phi_j^{\,2}(x)dx}{\int_L m(x)\phi_j^{\,2}(x)dx}=\frac{1}{m_{eq\_j}}$$

equivalent mass for j-th mode

$$S^t_{q_{y_j}}(\omega)=\lim_{t\to\infty}\frac{2\pi\cdot\hat{q}_{y_j}(\omega)\overline{\hat{q}_{y_j}(\omega)}}{T} \qquad S_{q_{y_j}}(\omega)=2S^t_{q_{y_j}}(\omega)$$

$$0<\omega<\infty$$

$$S_{q_{y_j}}(f)=\left\{\frac{\int_L \phi_j^{\,2}(x)dx}{\omega_j^{\,2}\int_L m(x)\phi_j^{\,2}(x)dx}\right\}^2\frac{1}{\left\{1-\left(\frac{f}{f_j}\right)^2\right\}^2+\left(\frac{\delta_j}{\pi}\right)^2\left(\frac{f}{f_j}\right)^2}S_{q_{p_j}}(f)$$

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PSD of generalized force q_{pj} for j-th mode

$$q_{p_j}(t)=\frac{\int_L p(t,x)\phi_j(x)dx}{\int_L \phi_j^2(x)dx}$$

$$S_{q_{p_j}}(f)=\frac{1}{\left\{\int_L \phi_j^2(x)dx\right\}^2}\iint_L S_p(x,x';f)\phi_j(x)\phi_j(x')dx$$

$$=\frac{1}{\left\{\int_L \phi_j^2(x)dx\right\}^2}\iint_L S_p(f)R_p(x,x';f)\phi_j(x)\phi_j(x')dxdx'$$

$$=\left|J_j(f)\right|^2 S_p(f)$$

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$$p(z,t)=\frac{1}{2}\rho\left(\bar{U}+u(z,t)\right)^2 C_D D$$

$$=\frac{1}{2}\rho\bar{U}^2 C_D(0)D+\rho\bar{U}u(z,t)C_D(f)D$$

$$=\bar{P}+2\bar{P}\frac{C_D(f)}{C_D(0)}\frac{u(z,t)}{\bar{U}}$$

$$p_{ref}(t)=\frac{\int_D p(z,t)dz}{D}$$

$$S_p(f)=S_{Pref}(f)=\frac{\iint_D S_p(z,z';f)dzdz'}{D^2}$$

$$=\frac{4\bar{P}^2}{D^2\bar{U}^2}\left\{\frac{C_D(f)}{C_D(0)}\right\}^2\iint_D R_u(z,z';f)dzdz' S_u(f)$$

$$=\frac{4\bar{P}^2}{\bar{U}^2}\left|\chi_A(f)\right|^2 S_u(f)$$

$$\therefore\ S_{q_{p_j}}(f)=\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)\frac{4\bar{P}^2}{\bar{U}^2}$$

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(continued)

$$S_{q_{y_j}}(f)=\frac{4\overline{P}^2\left\{\int_L \phi_j^{\,2}(x)dx\right\}^2}{\overline{U}^2\omega_j^{\,2}\left\{\int_L m(x)\phi_j^{\,2}(x)dx\right\}^2}\left|\chi_{d_j}(f)\right|^2\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)$$

ω_j : natural circular frequency of *j*-th mode [rad/s]

$m(x)$: mass per unit span length of a girder element [kg/m]

$\left|\chi_{d_j}(f)\right|^2$: dynamic magnification for *j*-th mode

$$\left|\chi_{d_j}(f)\right|^2=\frac{1}{\left\{1-\left(\frac{f}{f_j}\right)^2\right\}^2+\left(\frac{\delta_j}{\pi}\right)^2\left(\frac{f}{f_j}\right)^2}$$ ——★

f_j : natural frequency of *j*-th mode [Hz]

δ_j : structural + aerod. Damping for *j*-th mode (logarithmic damping decrement)

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(continued 2)

$$S_{q_{y_j}}(f)=\frac{4\overline{P}^2\left\{\int_L \phi_j^{\,2}(x)dx\right\}^2}{\overline{U}^2\omega_j^{\,2}\left\{\int_L m(x)\phi_j^{\,2}(x)dx\right\}^2}\left|\chi_{d_j}(f)\right|^2\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)$$

$\left|\chi_A(f)\right|^2$: aerodynamic admittance

$$\left|\chi_A(f)\right|^2=\frac{2}{(k\xi)^2}\{k\xi-1+\exp(-k\xi)\}$$ ——★

$$\xi=\frac{fD}{\overline{U}}$$

$$\left|\chi_A(f)\right|^2=\frac{1}{D^2}\left\{\frac{C_D(f)}{C_D(0)}\right\}^2\iint_D R_u(z,z';f)dzdz'$$

$$R_u(z,z';f)=\exp\left(-k\frac{f|z-z'|}{\overline{U}}\right)$$

Coherence of wind speed fluctuation component $u(x,z,t)$ along z direction

k : decay factor $\quad k=7$

$$\left\{\frac{C_D(f)}{C_D(0)}\right\}=1$$

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(continued 3)

$$S_{q_{y_j}}(f)=\frac{4\overline{P}^2\left\{\int_L \phi_j^2(x)dx\right\}^2}{\overline{U}^2\omega_j^2\left\{\int_L m(x)\phi_j^2(x)dx\right\}^2}\left|\chi_{d_j}(f)\right|^2\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)$$

$\left|J_j(f)\right|^2$: joint mode acceptance

$$\left|J_j(f)\right|^2=\frac{1}{\int_L \phi_j^2(x)dx}\iint_L R_p(x,x';f)\phi_j(x)\phi_j(x')dxdx'$$ —— ★

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(continued 4)

$$S_{q_{y_j}}(f)=\frac{4\overline{P}^2\left\{\int_L \phi_j^2(x)dx\right\}^2}{\overline{U}^2\omega_j^2\left\{\int_L m(x)\phi_j^2(x)dx\right\}^2}\left|\chi_{d_j}(f)\right|^2\left|\chi_A(f)\right|^2\left|J_j(f)\right|^2 S_u(f)$$

$S_u(f)$: PSD of wind speed fluctuation $u(t)$ [$m^2/s^2/Hz$]

$$S_u(f)=\frac{2K_r U_{10}^2}{f}\frac{X_D^2}{\left(1+X_D^2\right)^{\frac{4}{3}}}$$ (Davenport's formula) —— ★

$$X_D=\frac{1200f}{U_{10}}$$

K_r : surface roughness coefficient

ice, sea	0.003
grass	0.005
open area	0.008
trees, bush	0.015

U_{10} : average wind speed at 10m height [m/s]

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2nd semester, 2016
Earthquake and Wind resistance of Structures,
and Related Structural Design Principles

2) Characteristics of natural wind and Design wind load

1

Frictionless wind balance

◆Pressure gradient force $\frac{1}{\rho}\frac{\partial p}{\partial n}$ --(a)

◆Coriolis force $2\omega V \sin\phi$ --(b)

◆Centrifugal force $\frac{V^2}{r}$ --(c)

ρ : air density [kg/m^3]
ω : angular velocity of the earth [rad/sec]
V : wind speed [m/s]
ϕ : the latitude of the point [rad]

Per unit mass

2

Frictionless wind balance

Geostrophic wind: (a) = (b)
（地衡風）

$$\frac{1}{\rho}\frac{\partial p}{\partial n} = 2\omega V_{gs}\sin\phi \qquad \therefore\ V_{gs} = \frac{\frac{\partial p}{\partial n}}{2\rho\omega\sin\phi}$$

Gradient wind: (a) = (b) + (c)
（傾度風）

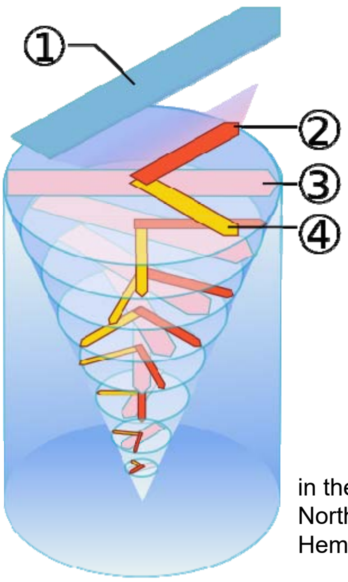
$$\frac{1}{\rho}\frac{\partial p}{\partial n} = 2\omega V_{gr}\sin\phi + \frac{V_{gr}^{2}}{r}$$

$$\therefore\ V_{gr} = -r\omega\sin\phi + \left[\frac{r}{\rho}\frac{\partial p}{\partial n} + (r\omega\sin\phi)^2\right]^{\frac{1}{2}}$$

in the Northern Hemisphere for cyclonic wind

3

Ekman spiral in atmospheric boundary layer



①Wind, ②Shear force from above layer, ③Effective direction of the current, ④Coriolis effect

in the Northern Hemisphere

4

Natural winds

- Normal wind
- Monsoon
- Typhoon (26.7 annually, 2.6 strike, 10.8 close)
- Tornado (18.7 annually, offshore)
- Local wind

always ↕ rarely

5

Weibull distribution

- Normal wind velocity (10 min. mean)

$$P(V^*) = \Pr[V > V^*] = \exp\left\{-\left(\frac{V^*}{c}\right)^k\right\}$$

Exceeding prob.

c : scale parameter [m/s]

k : shape parameter

6

Annual maximum wind speed

- N sample of size n from a mother dist. (n=8×365=2,920)
- Order the sample as $x_1 \leqq x_2 \leqq \ldots \leqq x_i \ldots \leqq x_n$
- The extrema (x_1 or x_n) will approach to the particular distribution, if n and N are large enough.

→ Fisher Tippet I, II, III

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Fisher-Tippet I, II, III type distribution

- Depends on the mother distribution

$$G_Y(y) = [F_x(y)]^n$$

$G_Y(y)$ Non-exceeding prob.

$$G_Y(y) = \int_{-\infty}^{y} g_Y(\xi) d\xi \,, \qquad g_Y(y) = \frac{dG_Y(y)}{dy}$$

$$g_Y(y) = \frac{dG_Y(y)}{dy} = n[F_x(y)]^{n-1} f_x(y)$$

A probability distribution $G_Y(y)$ of the extrema Y depends only on the upper tail of $F_x(x)$, not on the entire distribution of $F_x(x)$.

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$$G_Y(y) = [F_x(y)]^n$$

$G_Y(y)$: Non-exceeding prob. of the extrema (annual maximum wind speed) $\Pr[x_n \le y]$

$F_X(y)$: Non-exceeding prob. of the normal wind (every 3 hours wind speed record, $n = 8 \times 365 = 2{,}920$)

$\Pr[x_i \le y]$ $(i:$ any of $n)$

$$F_x(y) = 1 - P(y) = \Pr[V \le y] = 1 - \exp\left\{-\left(\frac{y}{c}\right)^k\right\}$$

(1-Weibull dist.)

Probability :
"none of n data doesn't exceed the maxima, x_n"

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Fisher Tippet I (Gumbel)

- Exponential type

$$F_X(x) = 1 - \exp\{-g(x)\}$$

$$G_Y(y) = \exp\{-\exp(-\alpha(y-u))\} \qquad -\infty \le y \le \infty$$

u: mode, α: shape parameter

$$m_Y = u + \frac{\gamma}{\alpha} \qquad \sigma_Y = \frac{\pi}{\sqrt{6}\alpha} \approx \frac{1.2825}{\alpha}$$

γ: Euler constant
$\gamma = 0.5772$

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Fisher Tippet II (Frechet)

- Polynomial type

$$F_X(x) = 1 - \beta\left(\frac{1}{x}\right)^k \qquad u\text{: mode, } k\text{: shape parameter}$$

$$G_Y(y) = \exp\left\{-\left(\frac{u}{y}\right)^k\right\} \qquad y \geq 0$$

$$m_Y = u\Gamma\left(1 - \frac{1}{k}\right) \quad k > 1 \qquad \sigma_Y{}^2 = u^2\left[\Gamma\left(1 - \frac{2}{k}\right) - \Gamma^2\left(1 - \frac{1}{k}\right)\right] \quad k > 2$$

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Fisher Tippet III (Weibull)

- Minima

$$F_X(x) = a(x - \varepsilon)^k$$

$$G_Y(y) = 1 - \exp\left[-\left(\frac{y - \varepsilon}{u - \varepsilon}\right)^k\right] \qquad y \geq \varepsilon$$

$$m_Y = \varepsilon + (u - \varepsilon)\Gamma\left(1 + \frac{1}{k}\right) \qquad \sigma_Y{}^2 = (u - \varepsilon)^2\left[\Gamma\left(1 + \frac{2}{k}\right) - \Gamma^2\left(1 + \frac{1}{k}\right)\right]$$

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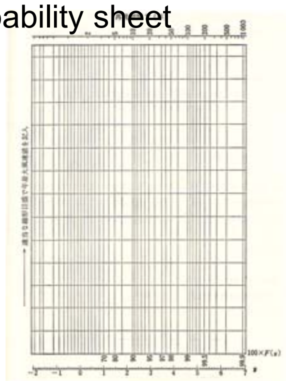
Which type will fit the observed annual maximum wind speed data best?

- Exceeding probability for each sample (empirical formula of exceeding probability)
- Plot on the double exponential probability sheet



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Empirical formula

$$P_j = \frac{j}{N}$$

$$P_j = \frac{j}{N+1}$$ (Gumbel) underestimate

$$P_j = \frac{j-1}{N}$$ → overestimate

$$P_j = \frac{2j-1}{2N}$$ (Hazen) → most practical

$$P_j = \frac{j-a}{N+1-2a}$$ (Gringorten) $a \fallingdotseq 0.44$ for $N \geqq 20$

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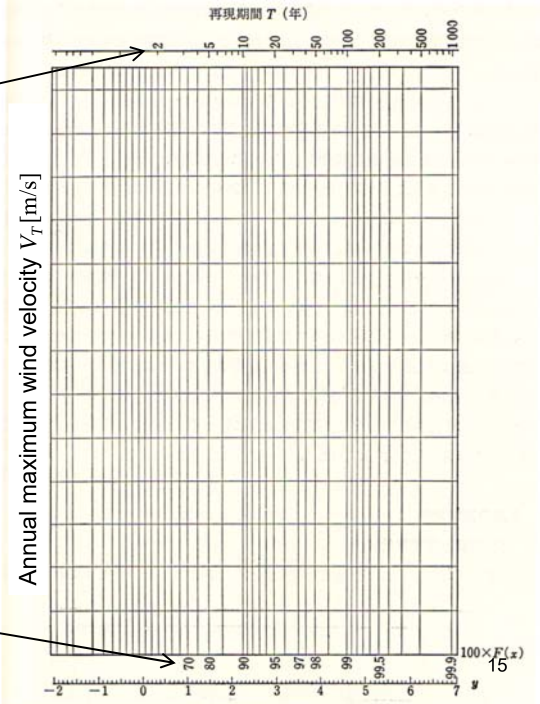
Double exponential probability sheet

Return period T [year]



Non-exceeding prob. $G_Y(y)$ [%]

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Annual maximum wind speed and return period

$$T = \frac{1}{1-q} = \frac{1}{1-G_Y(V_T)}$$

T: return period of wind speed V_T (year)

q: non-exceeding probability

$$V_T = u + \frac{1}{a}\ln T$$

For T>5

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Strong wind evaluation

$$G_Y(V) = \underline{\exp[-\exp\{-\alpha(V-u)\}]}$$

① determine distribution function (determine α, u)

$$G_Y(V) = 1 - \frac{1}{T}$$

② Determine return period T

(for road bridge : T=100 (yr))

③ value of V can be determined (V=40m/s)

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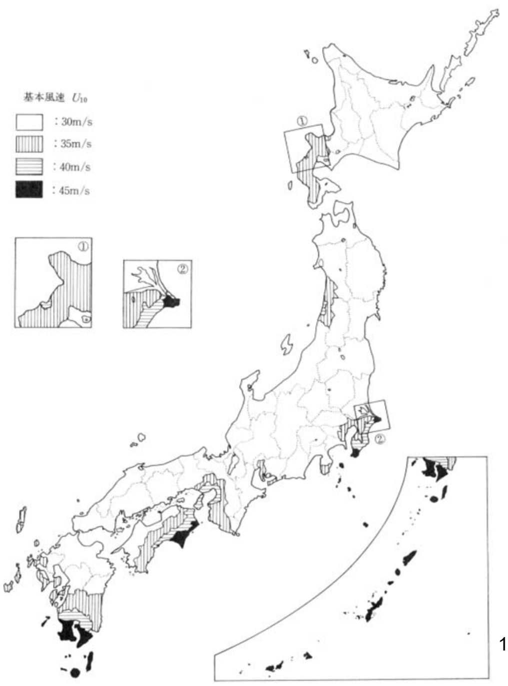
Design manual of wind resistant design for road bridges, 2005
(道路橋耐風設計便覧)
Japan road association
(日本道路協会)



18

Recommendations for loads on buildings Chap.6 Wind loading (2004, AIJ)

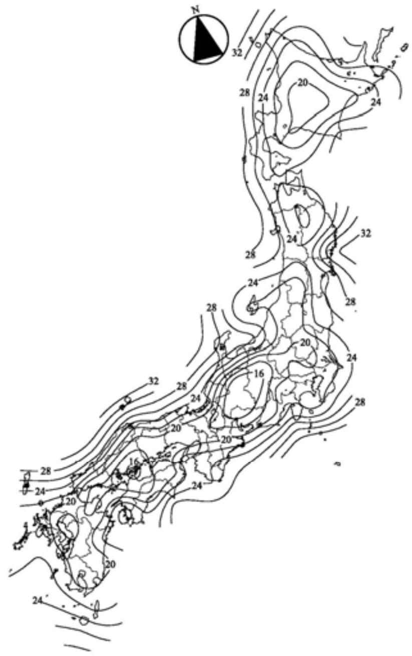


Figure A6.1.5 100-year-recurrence 10-minutes mean wind speed at 10m above ground over a flat and open terrain in winter (m/s)

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An example to determine design wind speed
- fitting to Gumbel dist. -

$$G_Y(y) = \exp\{-\exp(-\alpha(y-u))\}$$

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Annual maximum wind speed data

Year No.	Annual maximum wind speed (m/s)	Year No.	Annual maximum wind speed (m/s)
1	8.1	11	11.7
2	7.3	12	10.3
3	10.0	13	9.0
4	9.4	14	8.0
5	7.8	15	11.1
6	10.2	16	8.4
7	7.9	17	8.2
8	9.4	18	10.2
9	7.4	19	11.5
10	8.8	20	8.3

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Fisher Tippet I (Gumbel)

➢Exponential type

$$G_Y(y) = \exp\{-\exp(-\alpha(y-u))\} \qquad -\infty \le y \le \infty$$

u: mode, α: shape parameter

$$m_Y = u + \frac{\gamma}{\alpha} = 9.15\,[m/s] \qquad \sigma_Y = \frac{\pi}{\sqrt{6}\alpha} \approx \frac{1.2825}{\alpha} = 1.32\,[m/s]$$

$$\therefore \alpha = 0.97, \; u = 8.56$$

$$G_Y(y) = \exp\{-\exp(-0.97(y-8.56))\} \Leftrightarrow G_Y(y) = 1 - \frac{1}{T(y)}$$

y=13.29 [m/s] for 100yr., y=15.66 [m/s] for 1000yr.

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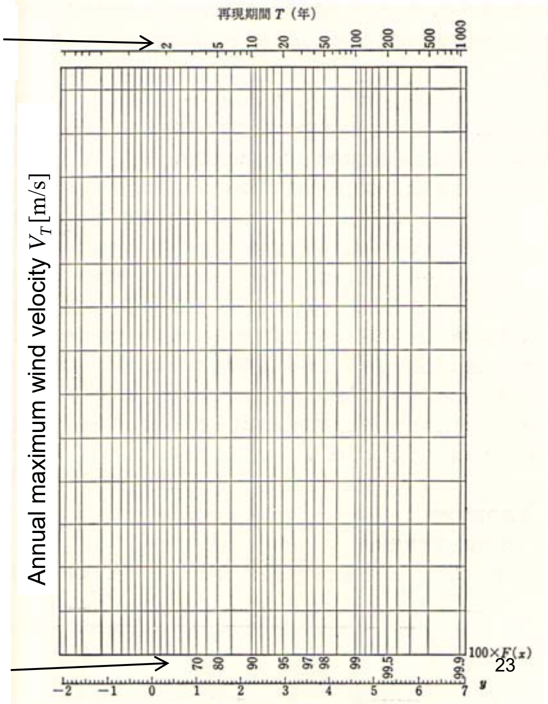
Return period
T [year]

● Double exponential probability sheet

● Hazen plot

$$P_j = \frac{2j-1}{2N} \quad (N=20)$$

(Exceeding prob.)



Non-exceeding prob. $G_Y(y)$ [%]

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Annual maximum wind speed data -ordering, P_j-

$$P_j = \frac{2j-1}{2N}$$

Data No. j	Annual maximum wind speed (m/s)	P_j	Data No. j	Annual maximum wind speed (m/s)	P_j
1	11.7	0.025	11	8.8	0.525
2	11.5	0.075	12	8.4	0.575
3	11.1	0.125	13	8.3	0.625
4	10.3	0.175	14	8.2	0.675
5	10.2	0.225	15	8.1	0.725
6	10.2	0.275	16	8.0	0.775
7	10.0	0.325	17	7.9	0.825
8	9.4	0.375	18	7.8	0.875
9	9.4	0.425	19	7.4	0.925
10	9.0	0.475	20	7.3	0.975

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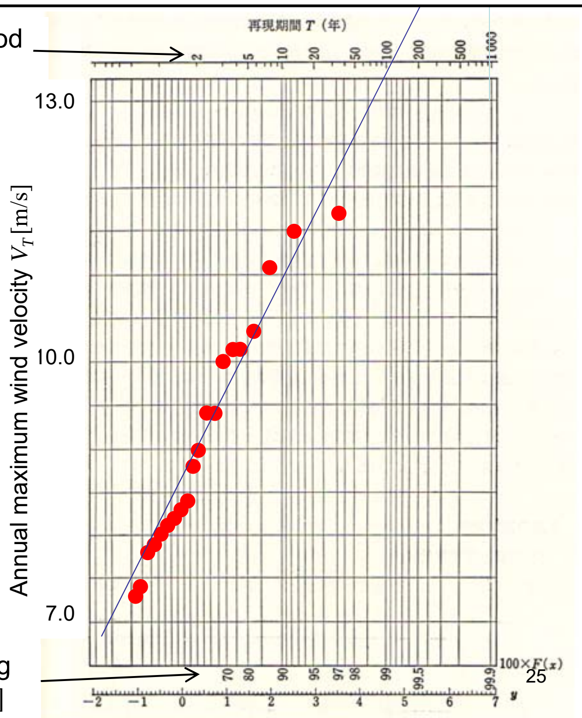
Return period
T [year]

Double exponential probability sheet

Hazen plot

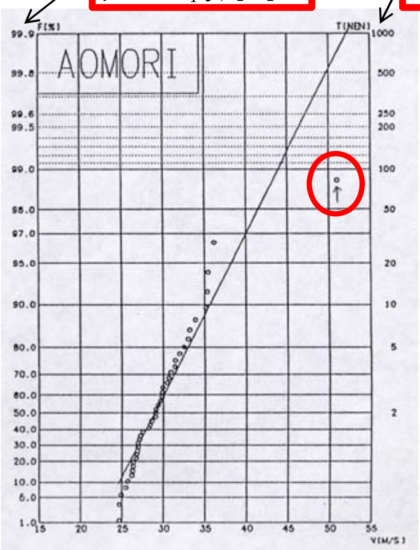
$$P_j = \frac{2j-1}{2N} \qquad (N=20)$$

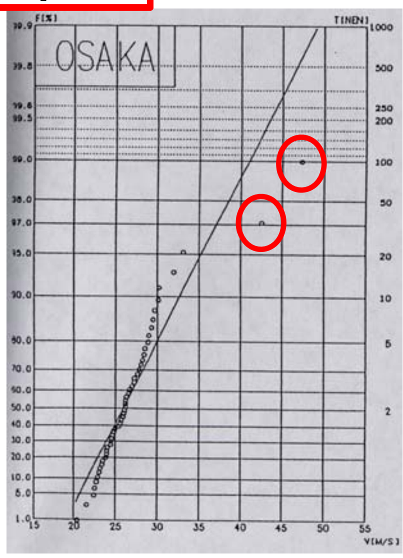


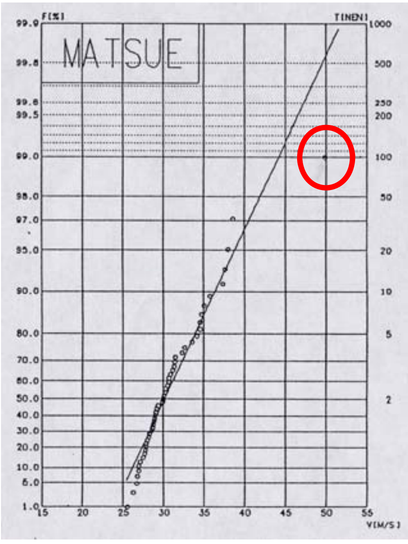
Non-exceeding prob. $G_Y(y)$ [%]

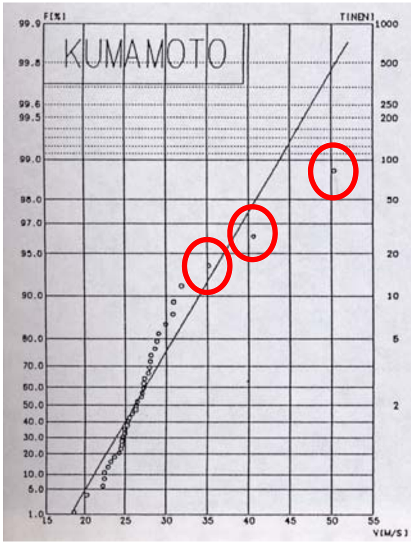
Non-exceeding prob. $G_Y(y)$ [%]

Return period T [year]









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Design wind load for road bridges

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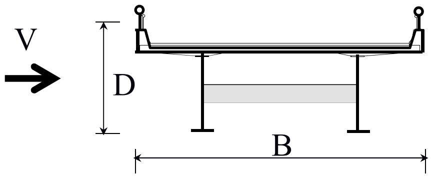
Gust Factor



$$P = \frac{1}{2}\rho V^2 C_D G A_n$$

The ratio of the maxima to the average of wind load due to wind turbulence and buffeting response, G=1.9

- P Wind load [N/m]
- ρ Air density [kg/m^3]
- V Design wind speed [m/s], V=40m/s
- C_d Drag force coefficient
- G Gust response factor
- A_n Effective project area [m^2/m]

Design code for Road bridges, 2012 (道路用示方書)
Japan Road Association
(日本道路協会)

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Gust Factor

Wind speed [m/s]

$$U(t) = \overline{U} + u(t)$$

$$U(t)\big|_{\max} \cong \overline{U} + 3\sigma_u = \overline{U}\left(1 + \frac{3\sigma_u}{\overline{U}}\right), \qquad Iu \equiv \frac{\sigma_u}{\overline{U}}$$

$$\Leftrightarrow \qquad G_U \equiv \frac{U(t)\big|_{\max}}{\overline{U}} = 1 + 3Iu$$

Wind load [N/m]

$$P(t) = \overline{P} + p(t), \qquad P \propto U^2 \qquad \Leftrightarrow \qquad G = G_U{}^2$$

$$G \equiv \frac{P(t)\big|_{\max}}{\overline{P}} = 1.9 \qquad \Leftrightarrow \qquad G_U = \sqrt{1.9} = 1.378 \quad \therefore Iu = 0.126$$

At 10m elevation
on the sea, open terrain

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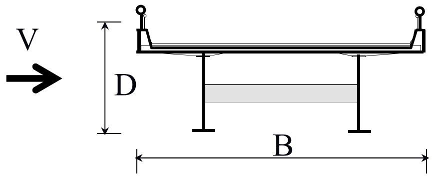
Drag force coefficient



$$P = \frac{1}{2}\rho V^2 C_D G A_n$$

Depends on structural geometry of cross section

P Wind load [N/m]

ρ Air density [kg/m^3]

V Design wind speed [m/s], V=40m/s

C_d Drag force coefficient

G Gust response factor

A_n Effective project area [m^2/m]

Design code for Road bridges, 2012 (道路用示方書)
Japan Road Association
(日本道路協会)

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Drag force coefficient Design code for Road bridges(日本道路協会)

Steel girder

$$C_d = \begin{cases} 2.1 - 0.1(B/D) & \cdots\cdots \quad (1 \le B/D < 8) \\ 1.3 & \cdots\cdots \quad (8 \le B/D) \end{cases}$$

2 plane truss

$$C_d = \begin{cases} 1.35/\sqrt{\phi} & (0.1 \le \phi \le 0.6) \quad \text{for truss} \\ 1.6 & \text{for deck} \end{cases}$$

ϕ : solidity ratio

Arch (angular member)

$$C_d = \begin{cases} 1.6 & \text{for windward} \\ 0.8 & \text{for leeward} \end{cases}$$

Arch (pipe member)

$$C_d = 0.8$$

Suspension, Cable-stayed
(steel girder, 2 plane truss)

C_d (see above)

(tower)

$$C_d = \begin{cases} 1.6 & \text{for wind ward} \\ 0.8 & \text{for leeward} \end{cases}$$

(cable)

$$C_d = 0.8$$

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Hurricane simulation

Journal of Wind Engineering and Industrial Aerodynamics, 13 (1983) 139—152 139
Elsevier Science Publishers B.V., Amsterdam — Printed in The Netherlands

DESIGN WIND SPEEDS IN REGIONS DOMINATED BY TROPICAL CYCLONES

P. N. Georgiou, A. G. Davenport and B. J. Vickery
Boundary Layer Wind Tunnel Laboratory, The University of Western Ontario,
London, Ontario, Canada

SUMMARY

In recent years computer simulation methods predicting extreme wind speeds in regions dominated by tropical cyclones have been developed and shown to give reasonable estimates of long-term risk levels arising from these winds. In this paper refinements of the model are presented with the emphasis on improvements to the statistical distribution functions describing tropical cyclone characteristics used in the simulation. In particular the relationships chosen for the two parameters which influence wind speeds the most, the central pressure difference and the decay rate after landfall, represent significant changes from previous models. The application of the simulation to specific oceanic regions is discussed and the results of a case study conducted along the Gulf and Atlantic Coasts of the United States presented.

NOTATION

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3.2 Occurrence - Spatial Domain

Several methods exist for locating tropical cyclone paths with respect to any geographical point. The one chosen for the present simulation is shown in Figure 1. Tracks are simulated as straight line paths, defined by their approach angle, θ, taken clockwise positive from North, and their minimum approach distance, d_{min}. When a track passes to the left of a station of interest d_{min} is taken to be positive. In practice tropical cyclone tracks are never straight. There is however no way to model the erratic upper-level atmospheric conditions upon which the real curvature of observed paths depends. Moreover by limiting simulation circles

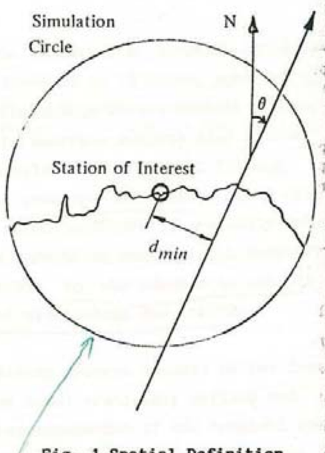

Fig. 1 Spatial Definition of Tropical Cyclone Tracks

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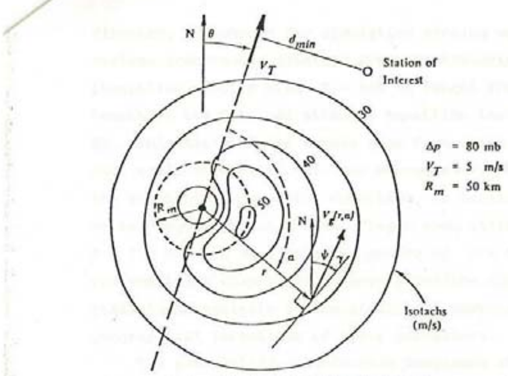

Fig. 2 Gradient Height Wind Field Simulated in Tropical Cyclone Model

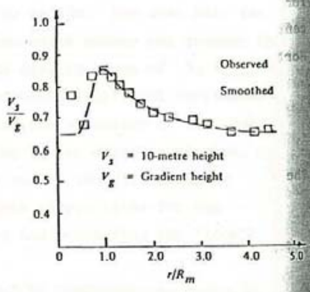

Fig. 3 Ratio of Surface to Gradient Wind Speeds Used in the Simulation

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TABLE 2

		Parameter	Probability Distribution Function Representation Used in the Simulation
λ	=	Annual Occurrence Rate	Poisson
d_{min}	=	Minimum Approach Distance	Polynominal
θ	=	Approach Angle	von Mises
Δp	=	Central Pressure Difference	Weibull
R_m	=	Radius of Maximum Winds	Log-normal conditionally dependent upon Δp
V_T	=	Translation Velocity	Log-normal

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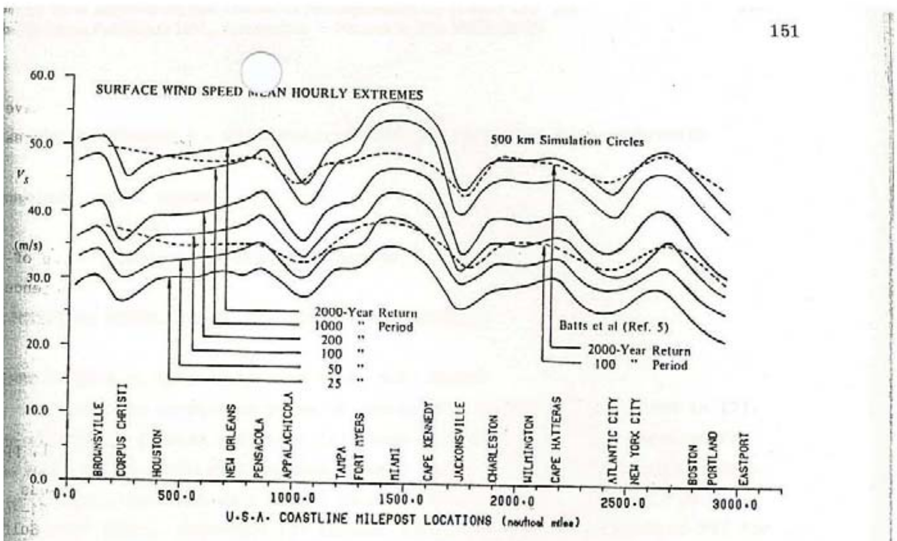


ig. 9 U.S.A. Coastline Variation of Mean Hourly Surface Wind Speed Extremes

37

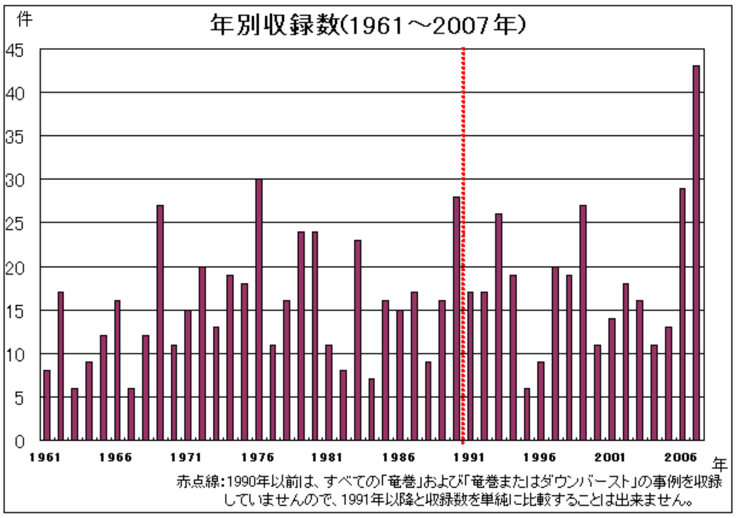
Extreme winds
(Tornadoes, downburst, gust front)

38

Extreme winds (Tornadoes, downburst, gust front

39

Annual number of tornado



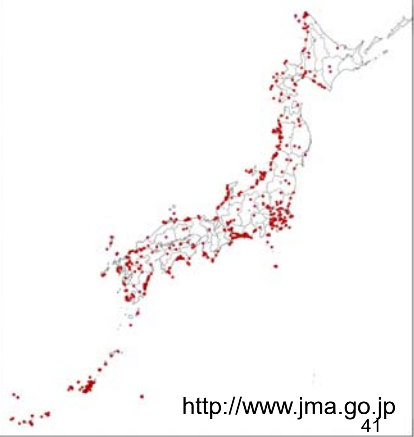
40

Extreme winds (Tornadoes, downburst, gust front)

- 18.66 annually (leads to disaster)
- Up to F3 (less than 90 m/s)
  (JMA statistics)
- 1.4 annually per 10^4 km^2 (comparable with in Alabama and Missouri) (Niino et al., 1997)
- 8 tornaodes annually may cross railway tracks anywhere in Japan (Tamura, 2007)

- Wind loads, flying debris
- Safety level for high hazardous facilities, mass-, high-speed transports

- Hard to be predicted



http://www.jma.go.jp

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Recent big tornadoes in Japan

表 11　FPP で整理した主な竜巻の比較

竜巻	茂原 1990.12.11	豊橋 1999.09.24	佐賀 2004.06.27	延岡 2006.09.17	佐呂間 2006.11.07
Life time (min)	7	25	7	5	3?
F scale	F3	F3	F2	F2	F3
P scale (length)	P2 6.5km	P3 19 km	P2 8 km	P2 7.5 km	P1? 1.5 km
P scale (width)	P3 500 m	P3 550 m	P2 200 m	P2 250 m	P2? 300 m

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Tornado disaster
6, May, 2012 in Tsukuba

https://www.city.tsukuba.ibaraki.jp/1330/010537.htm

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Fujita scale

Maximum wind speed

F scale	Description	Wind speed (m/s)	Damage to wooden living houses
F0	Mild	17-32	Light, local
F1	Weak	33-49	Roof tiles will be flying away.
F2	Strong	50-69	Roofs will be teared off.
F3	Severe	70-92	Houses will be collapsed.
F4	Violent	93-116	Houses will be totally dismantled.
F5	Unimaginable	117-142	Houses will dissapear.

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Design-basis tornado

(U.S. Nuclear Regulatory Commission. 2007. “Regulatory Guide 1.76”)

Table 1. Design-Basis Tornado Characteristics

Region	Maximum wind speed m/s (mph)	Translational speed m/s (mph)	Maximum rotational speed m/s (mph)	Radius of maximum rotational speed m (ft)	Pressure drop mb (psi)	Rate of pressure drop mb/s (psi/s)
I	103 (230)	21 (46)	82 (184)	45.7 (150)	83 (1.2)	37 (0.5)
II	89 (200)	18 (40)	72 (160)	45.7 (150)	63 (0.9)	25 (0.4)
III	72 (160)	14 (32)	57 (128)	45.7 (150)	40 (0.6)	13 (0.2)

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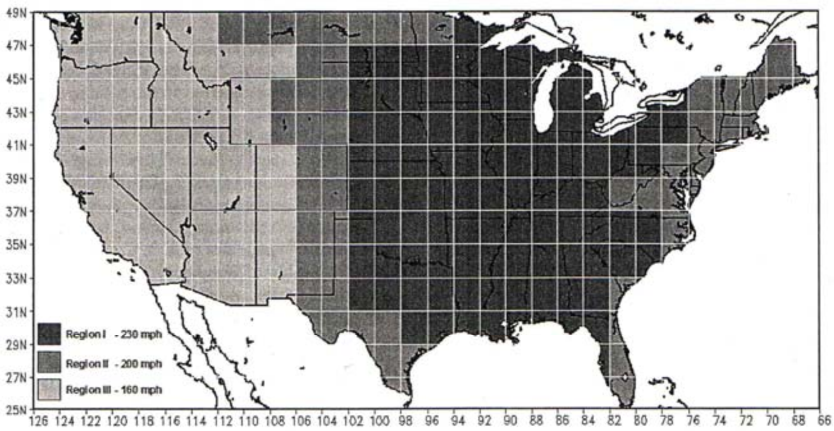

Figure 1. Tornado intensity regions for the contiguous United States for exceedance probabilities of 10^{-7} per year

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Flying debris

Table 2. Design-Basis Tornado Missile Spectrum and Maximum Horizontal Speeds

Missile Type		Schedule 40 Pipe	Automobile	Solid Steel Sphere
Dimensions		0.168 m dia × 4.58 m long (6.625 in. dia × 15 ft long)	Region I and II 5 m × 2 m × 1.3 m (16.4 ft x 6.6 ft x 4.3 ft) Region III 4.5 m x 1.7 m x 1.5 m (14.9 ft x 5.6 ft x 4.9 ft)	2.54 cm dia (1 in. dia)
Mass		130 kg (287 lb)	Region I and II 1810 kg (4000 lb) Region III 1178 kg (2595 lb)	0.0669 kg (0.147 lb)
C_DA/m		0.0043 m^2/kg (0.0212 ft^2/lb)	Region I and II 0.0070 m^2/kg (0.0343 ft^2/lb) Region III 0.0095 m^2/kg (0.0464 ft^2/lb)	0.0034 m^2/kg (0.0166 ft^2/lb)
V_{Mh}^{max}	Region I	41 m/s (135 ft/s)	41 m/s (135 ft/s)	8 m/s (26 ft/s)
	Region II	34 m/s (112 ft/s)	34 m/s (112 ft/s)	7 m/s (23 ft/s)
	Region III	24 m/s (79 ft/s)	24 m/s (79 ft/s)	6 m/s (20 ft/s)

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Recent research topics

(Aerodynamics under gusty wind condition)

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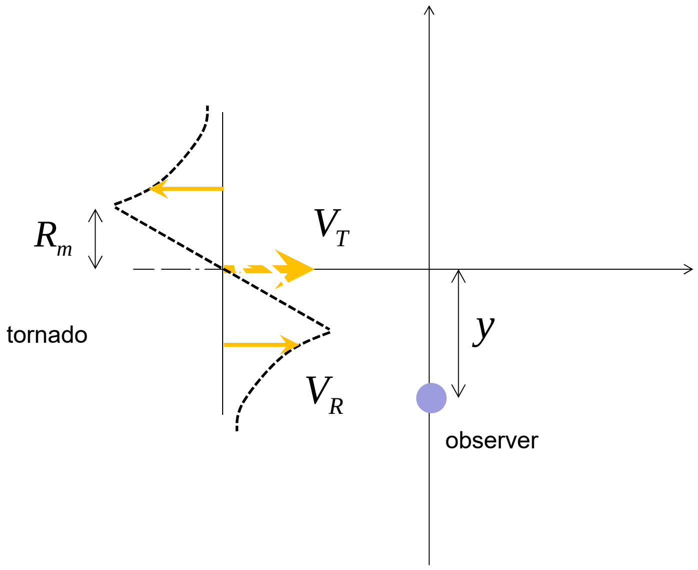
Severe wind by single tornado



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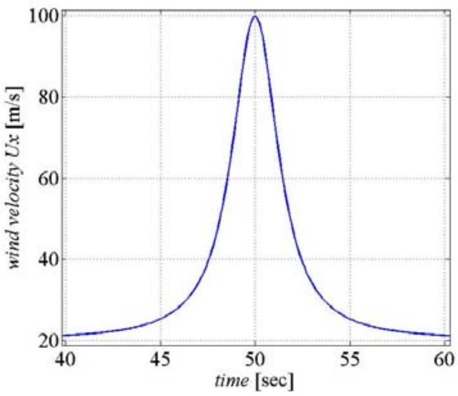
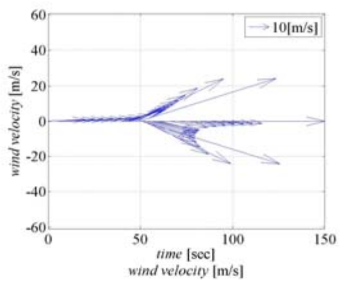
$U_x(t)$ component



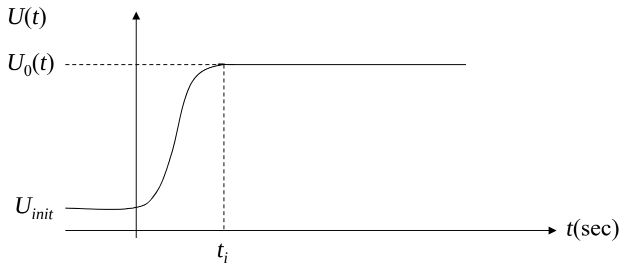


50

Gust, strong wind by tornado passage

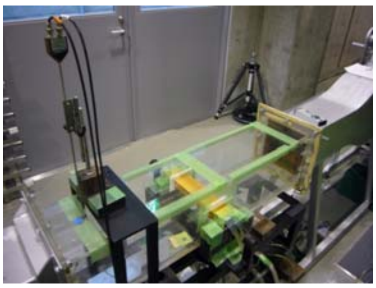
- Velocity increase in short time duration
  → like a step function

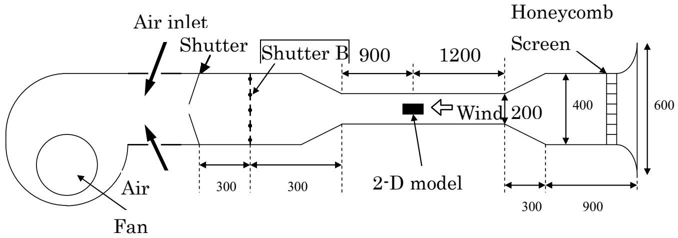
$$U(t) \approx U_0 \cdot 1(t) + U_{init}$$



51

Wind tunnel for short-time velocity increase





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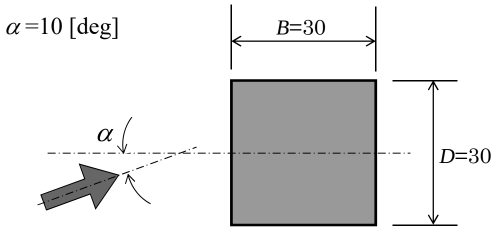
A square prism with incidence angle α=10 [deg]



U=3.0m/s to 7.0m/s, Re= 6.0x10^3 to 1.4x10^4

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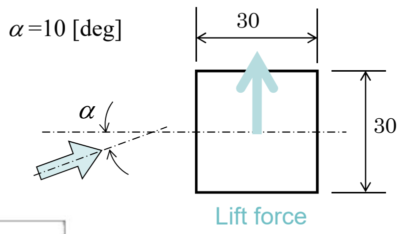

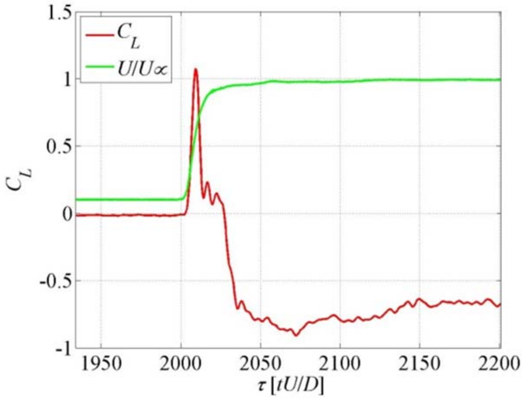

$$C_L = \frac{Lift\ \ force}{\frac{1}{2}\rho U_\infty^2 D\ell}$$

$$\tau = \frac{U_\infty t}{D}$$

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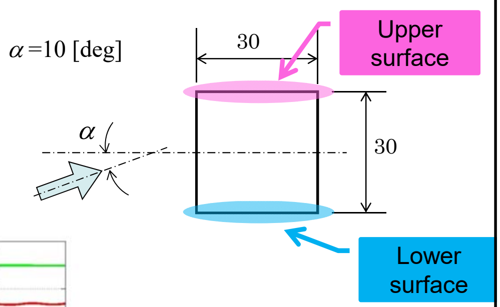

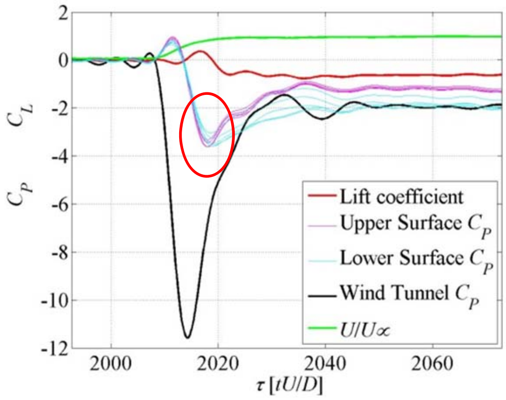

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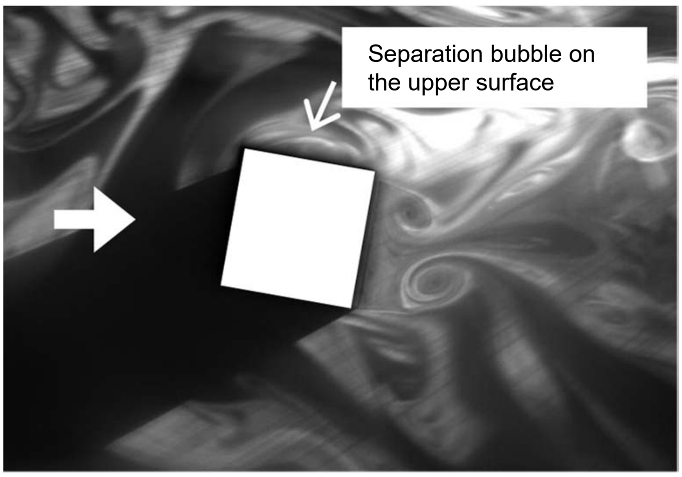
Flow visualization



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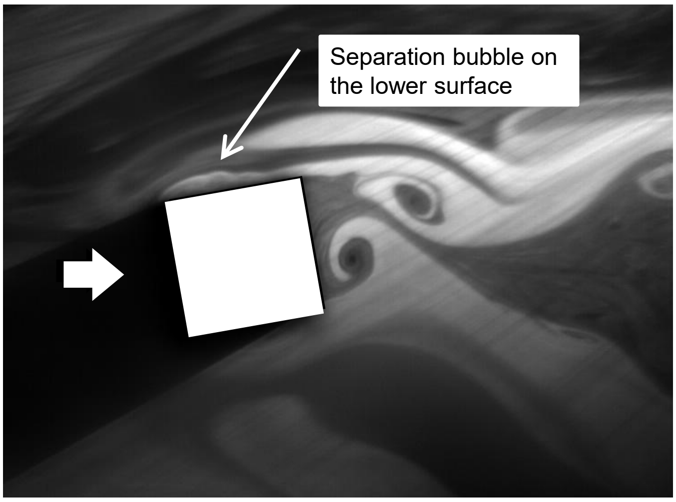
Flow visualization



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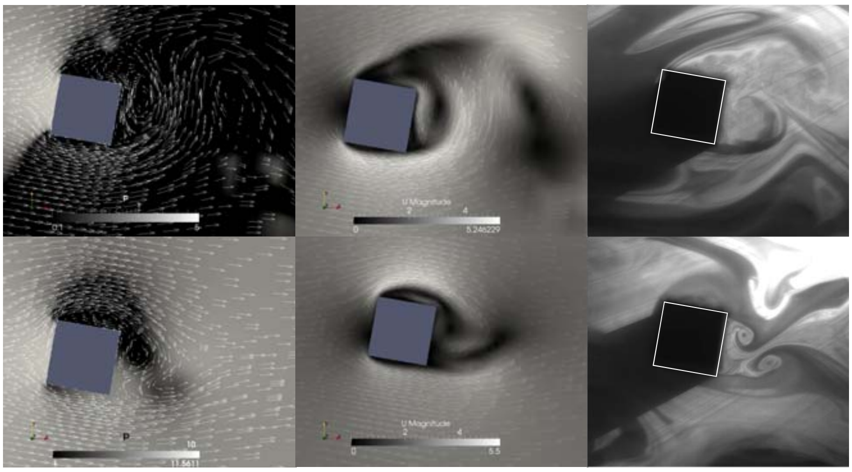
Drag force peak

Lift force peak



CFD
(with pressure contour)

CFD
(velocity contour)

Experiment

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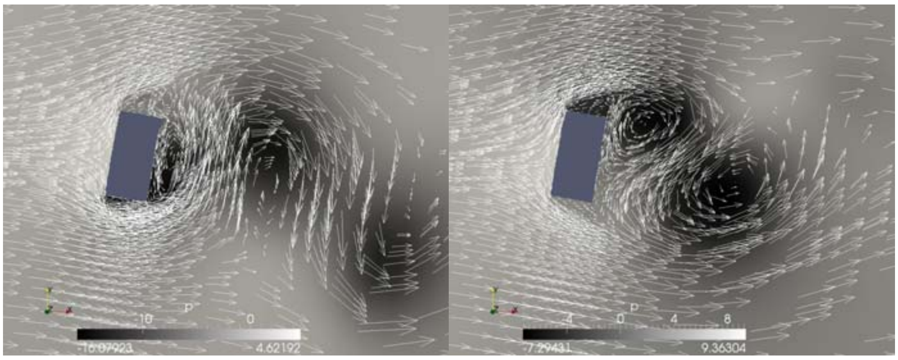
Rectangular prism with B/D=0.5



at drag peak　　　　at lift peak

(with pressure contour)

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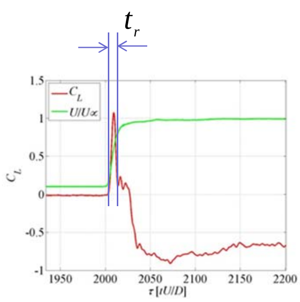
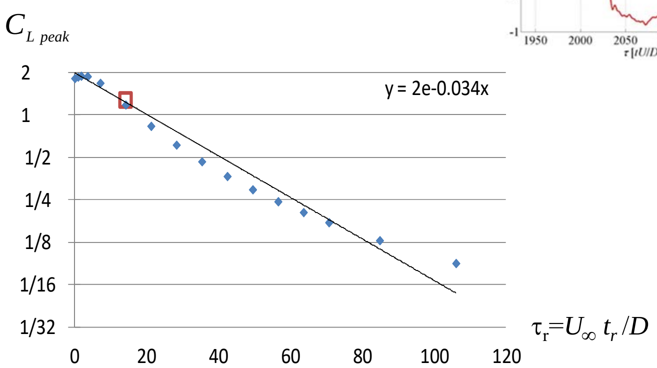
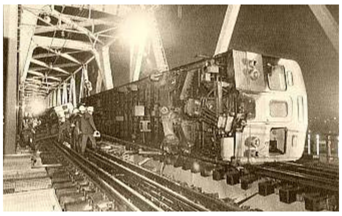
If t_r is relatively short, the peak value $C_{L\,peak}$ may exceed its steady state value (U=U_∞)





60



Feb.28, 1978 Tozai line

http://www.ne.jp/asahi/aluminium/mania/al_train/t_eidan_clash2.htm

28 Dec., 1986 San-in main line

http://www.sydrose.com/creativedesignengine/HTML/bb4-01213/bb4-01213.html

61

3 Apr., 2012 Niigata pref. http://livedoor.blogimg.jp/joetsutj/imgs/3/8/3811c96a.jpg

http://minkara.carview.co.jp/userid/143276/blog/25999634/

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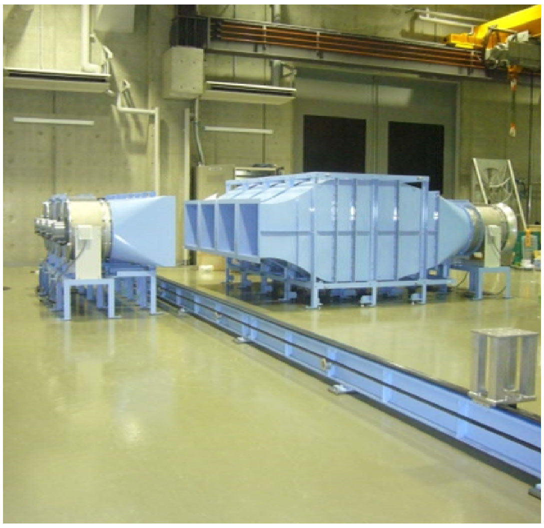
Vehicles overturning phenomenon due to lateral wind



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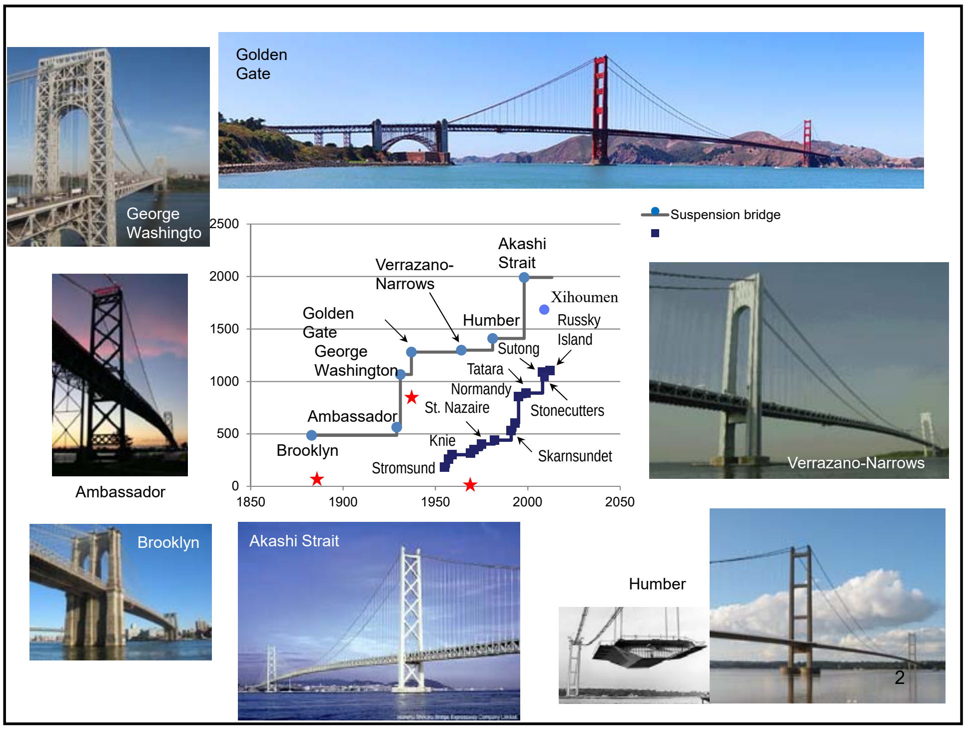
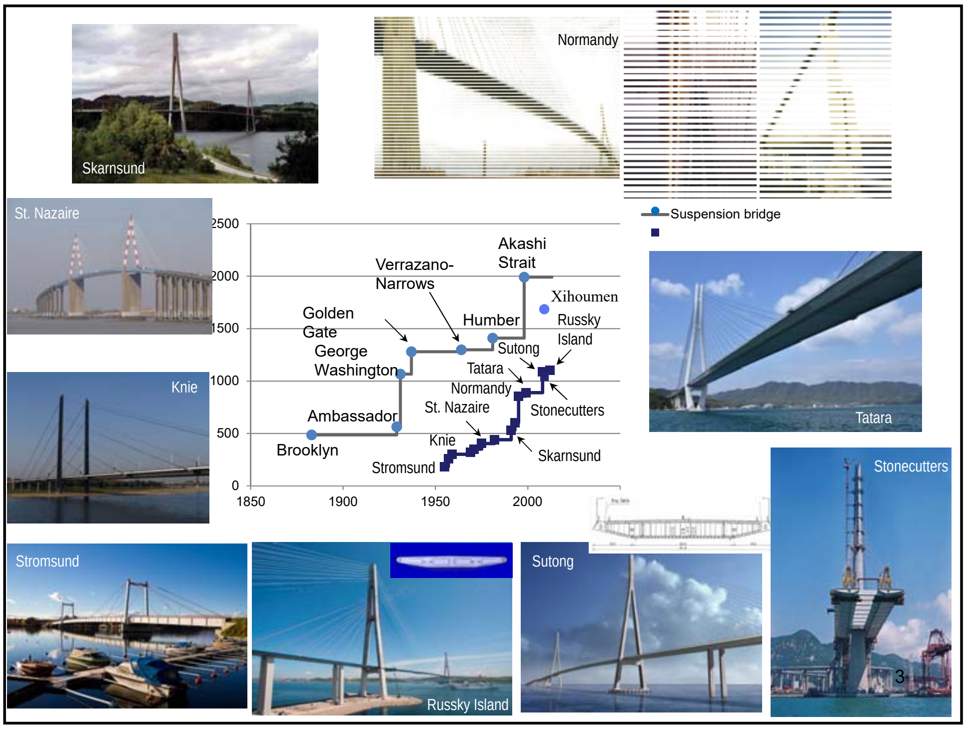
2nd semester, 2016
Earthquake and Wind resistance of Structures,
and Related Structural Design Principles

3) Wind resistance design codes

1

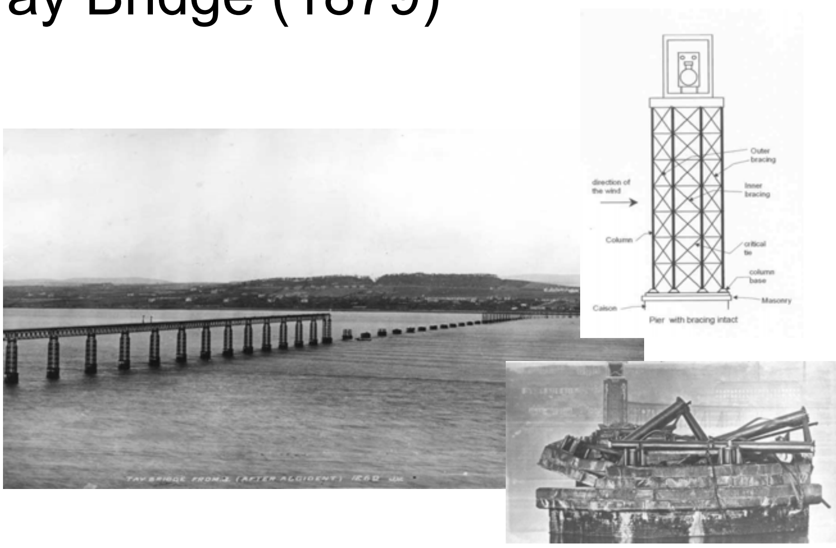


2



Tay Bridge (1879)

Learned by wind
Pioneers' footprint

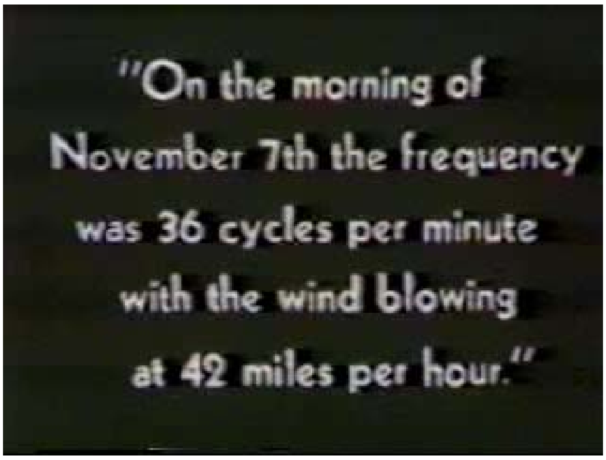


http://taybridgedisaster.co.uk/index/wind-theory

4

Learned by wind
Pioneers' footprint

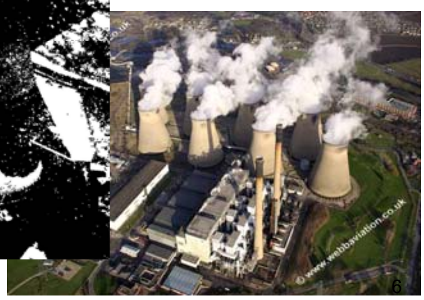
Old Tacoma Narrows Bridge (1940)



5

Learned by wind
Pioneers' footprint

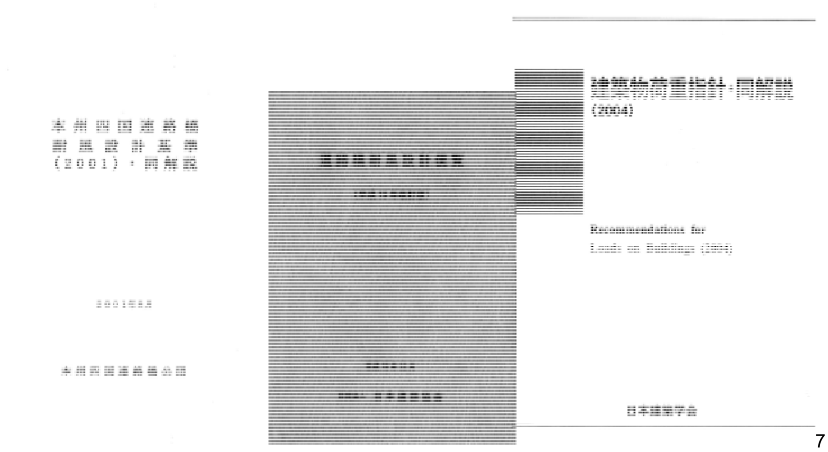
Ferrybridge Cooling Tower (1965)



6

Learned by wind
Pioneers' footprint

Wind resistant design codes



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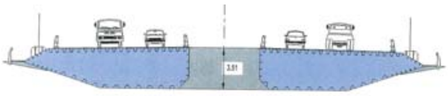
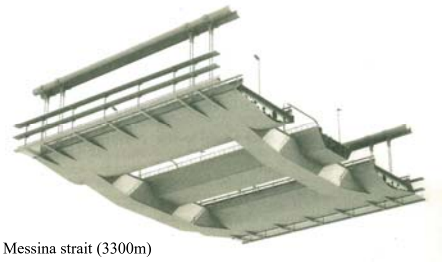
Learned by wind
Pioneers' footprint

Wind tunnel tests

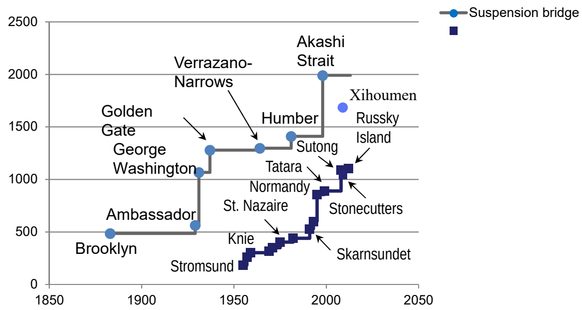
8



Xihoumen



Messina strait (3300m)



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Major codes in Japan for wind resistant design and wind loading

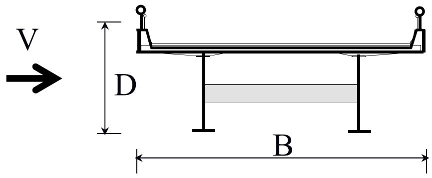
•Design code for Road bridges in Japan, 2012
(道路橋示方書･同解説) *Japan road association, JRA1*

•Wind resistant design standard, 2001
(本州四国連絡橋耐風設計基準) *Honshu-Shikoku Bridge Authority*

•Design manual of wind resistant design for road bridges, 2005
(道路橋耐風設計便覧） *Japan road association, JRA2*

10

(JRA1)



$$P = \frac{1}{2}\rho V^2 C_D G A_n$$

The ratio of the maxima to the average of wind load due to wind turbulence and buffeting response

Depends on structural geometry of cross section

- P Wind load [N/m]
- ρ Air density [kg/m³]
- V Design wind speed [m/s]
- C_d Drag force coefficient
- G Gust response factor
- A_n Effective project area [m²/m]

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Drag force coefficient (JRA1)

Steel girder

$$C_d = \begin{cases} 2.1 - 0.1(B/D) & \cdots\cdots \quad (1 \le B/D < 8) \\ 1.3 & \cdots\cdots \quad (8 \le B/D) \end{cases}$$

2 plane truss

$$C_d = \begin{cases} 1.35/\sqrt{\phi} & (0.1 \le \phi \le 0.6) \quad \text{for truss} \\ 1.6 & \text{for deck} \end{cases}$$

ϕ : solidity ratio

Arch (angular member)

$$C_d = \begin{cases} 1.6 & \text{for windward} \\ 0.8 & \text{for leeward} \end{cases}$$

Arch (pipe member)

$$C_d = 0.8$$

Suspension, Cable-stayed (steel girder, 2 plane truss)

C_d (see above)

(tower)

$$C_d = \begin{cases} 1.6 & \text{for wind ward} \\ 0.8 & \text{for leeward} \end{cases}$$

(cable)

$$C_d = 0.8$$

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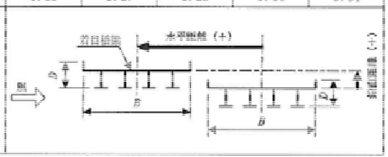
Modification factor of Drag force coefficient due to closely spaced arrangements

(JRA1)

表-3.3 風洞試験による並列橋の抗力の補正係数

鉛直距離 \ 水平距離	2B	B	0	−B	−2B
2D	0.94	1.25	1.14	1.12	0.98
D	0.94	1.04	1.39	0.56	0.66
0	0.97	1.14	−	−0.16	0.14
−D	0.93	1.10	1.65	0.54	0.61
−2D	0.93	1.27	1.28	1.14	0.94
備考					

注) 補正係数＝並列橋としての抗力/単独橋としての抗力
この補正係数は迎角が−3°から+3°の範囲の最大値である。

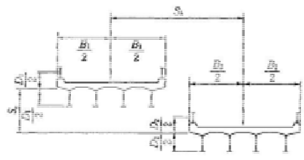


図-3.10 並列橋の位置関係

Wind resistant design procedure (HonShi)

1) Static design.
2) Wind tunnel test using a spring support section model to find aerodynamically stable geometry of a girder. Wind tunnel test using a full elastic model to find aerodynamically stable geometry of a tower.
3) Safety check for instability due to static wind load.
4) Safety check for divergent vibration, buffeting, vortex-induced vibration by wind tunnel test (full elastic model), flutter analysis and buffeting analysis.

Static effect
•Deformation, stress due to static wind load
•instability (lateral buckling, divergence)

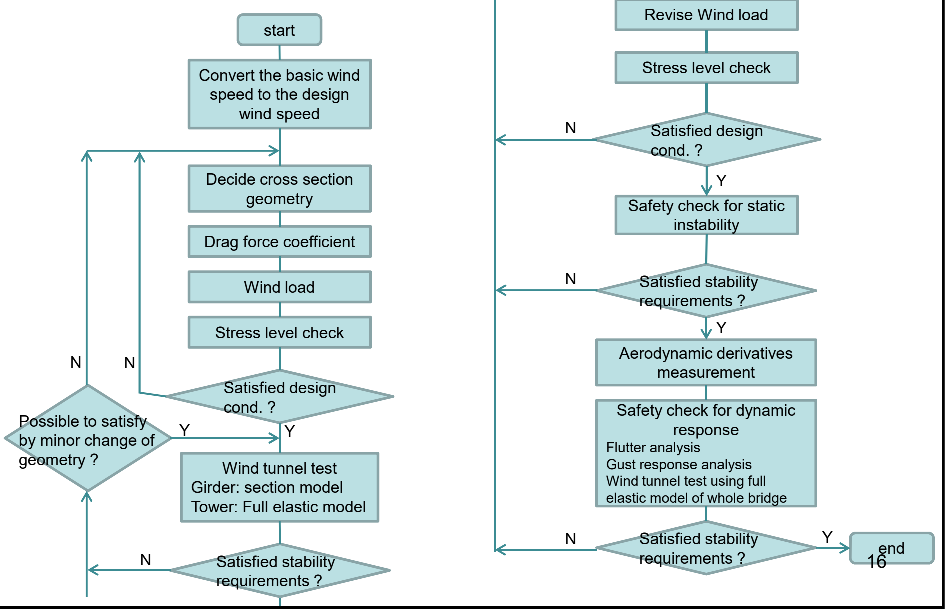
Dynamic effect
•Divergent vibration (Galloping, Flutter)
•Buffeting
•Vortex-induced vibration
•Cable vibration

Aerodynamic phenomena and structural elements to be assured its safety (HonShi)

			Girder	Tower/ Pylon	Main cable	Hanger	Cable for c.s.b.
Static effect	Deformation, stress		○	○	○	○	○
	instability	Lateral buckling	○	—	—	—	—
		divergence	○	—	—	—	—
Dynamic effect	Divergent vibration		○	○	—	—	—
	Buffeting		○	△	—	—	—
	Vortex-induced vibration		○	○	—	△	○
	Cable vibration		—	—	—	△	○

15

Wind-resistant design procedure for Akashi strait and Kurushima strait Bridges (HonShi)



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Basic wind speed $\bar{U}_{10}$ (HonShi)

Bridge name	Basic wind speed (m/s)
Akashi strait	46
Kurushima 1st, 2nd, 3rd	40
Tatara	37

History of the BSW for Akashi strait Bridge

- Wind observation at Tarumi obs. Tower (H=80m)
- 49.4m/s for 150 year return priod
- Bad fit to Gumbel dist. (Maximum ever recorded and 2nd maximum)
- 49.4 × 1.1=54.3m/s
- Wind velocity ratio of at Tarumi and at Akashi strait = 1:1.1 (by topographic model)
- Vertical profile 54.3 × 1.1 × (10/80)$^{1/8}$=45.9m/s

※ power α
: 1/8 (Akashi st. br.)
: 1/7 (Tatara br., Kurishima st. br.)

$$\bar{U}_z = \bar{U}_{10} \cdot \left(\frac{z}{10}\right)^{\alpha}$$

17

Gust response factor μ_2, μ_3 (HonShi)

Girder, cable, hanger, Suspended structures
$$P_D = \mu_2 \frac{\rho \bar{U}_z^{\,2}}{2} C_D A_n$$

Tower/pylon
$$P_D = \mu_3 \frac{\rho \bar{U}_z^{\,2}}{2} C_D A_n$$

Air density
$$\rho = 1.2 \quad [\mathrm{kg/m^3}]$$

Structural element to design			girder		tower	
Wind loading direction			Normal to bridge axis	Along bridge axis	Normal to bridge axis	Along bridge axis
Akashi strait	Main cable	μ_2	1.55	--	1.35	--
	Hanger rope Suspended girder	μ_2	1.55	1.25	1.35	1.25
	Tower	μ_3	--	--	1.55	1.50 (top fixed) 1.75 (top free)
Kurushima 1st, 2nd, 3rd	Main cable Hanger rope Suspended girder	μ_2	2.0	1.3	1.7	1.3
	Tower	μ_3	--	--	1.4	1.55 (top fixed) 1.75 (top free)
Tatara	Cable Main girder	μ_2	1.9	1.35	1.65	1.35
	Pylon	μ_3	1.9	1.5	1.65	1.5 (top fixed) 1.8 (top free)

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Gust response factor (HonShi)

μ_2, μ_3

$= G$ (JRA1)

depends on

●Spanwise correlation of wind velocity fluctuation

●Dynamic properties (Nat. freq., Damping, Vib. modes, etc.)

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Drag force coefficient (HonShi)

Structural element		Normal to bridge axis		Along bridge axis	
		C_D	remarks	C_D	remarks
girder	Truss structure	By wind tunnel test using similar geometry of cross section		60% to the value for normal to bridge axis	
	Box	By wind tunnel test using similar geometry of cross section		30% to the value for normal to bridge axis	
Tower		1.8 or	Per one column with rectangular cross section	1.8 or	Per column with rectangular cross section
		by wind tunnel test using similar geometry of cross section		by wind tunnel test using similar geometry of cross section	
Main cable for suspension bridge		0.7		--	
Hanger rope for suspension bridge		0.7		0.7/√2 0.7	For truss
Cable for cable-stayed bridge		0.7		0.7	

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Projected area A_n (HonShi)

Structural element	Normal to bridge axis	Along bridge axis
Girder	Windward area of girder, handrail and parapet	Same as left
Tower	Windward and leeward area of columns	Windward and leeward area of columns and bracing
Main cable for suspension bridge	Windward and leeward cables	--
Hanger rope for suspension bridge	Windward and leeward hanger ropes	Same as left
Cable for cable-stayed bridge	Windward and leeward cables	Same as left

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Basic wind speed during construction (HonShi)

Bridge name	Basic wind speed during construction (m/s)
Akashi strait	37
Kurushima 1st, 2nd, 3rd	31
Tatara	29

(Akashi) Durable period (R) =5(yr.), Non-exceding prob. (q) =0.8
→ Return period (T) =23(yr.)

(Kurishima, Tatara)
R=4, q=0.8 → T=18

$$q = \left(1 - \frac{1}{T}\right)^R$$

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Structural damping (Wind tunnel test, Flutter and buffeting analysis) (HonShi)

Structural part		Logarithmic damping decrement	
		Heaving vibration	Torsional vibration
Girder dominant mode	Truss stiffened girder	0.03	0.02 0.03 for main span length being 500 to 600m
	Box girder	0.02	0.02
Tower dominant mode	Tower, cable, girder system	0.02	0.02
	Isolated tower	0.01	0.01
Cable dominating mode for cable-stayed bridge		0.003	--
Hanger dominating mode for suspension bridge		0.003	--

Vertical inclination of wind incidence angle (HonShi)

- Smooth flow -3° ～ +3°
- Turbulent flow 0°

Maximum expected angle during the time duration being required for growth up of divergent vibration

Safety assurance for divergent vibration (HonShi)

For the critical wind velocity of divergent vibration U_F, the following criteria must be satisfied:

$$U_F \geq 1.2 \cdot \mu_F \cdot \overline{U}_z$$

Where, μ_F is a correction factor with regard to wind velocity fluctuation and $\overline{U}_z$ is the basic design wind speed.

Bridge name	μ_F
Akashi strait	1.08
Kurushima 1st, 2nd, 3rd	1.1
tatara	1.1

μ_F: The ration of the maximum expected wind speed in the duration of 5 times of natural period of divergent vibration

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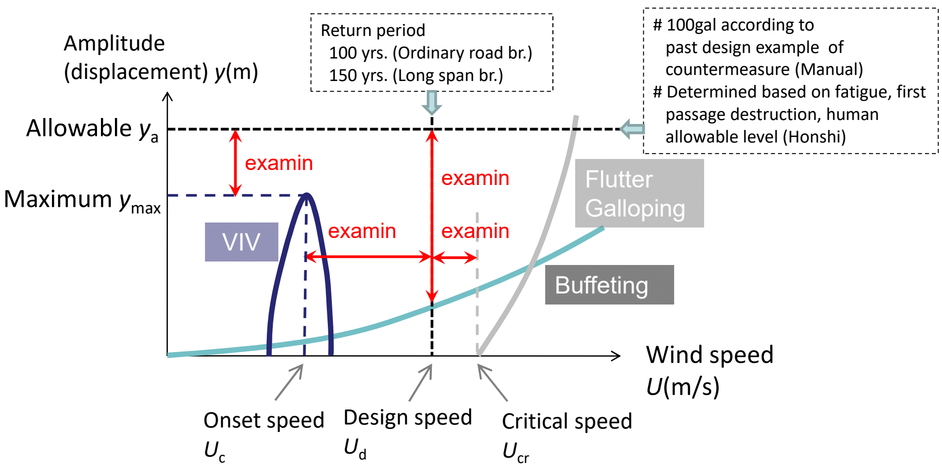
Examination of Aerodynamic stability (JRA2)



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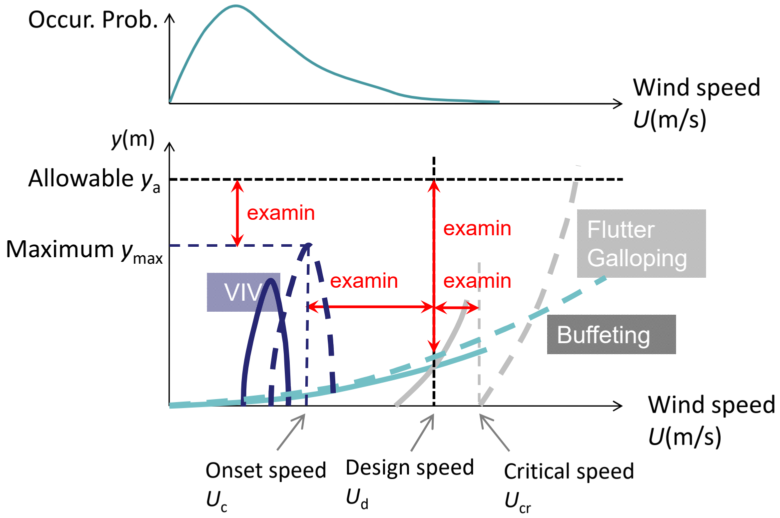
Examination of Aerodynamic stability

(JRA2)



If span length becomes longer…
→ Aerodynamic phenomena occur in lower wind speed.
= more frequent occurrence

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Criteria for aerodynamic phenomena

(JRA2)

- flutter

$$U_{cf} > U_{rf}$$
$$U_{cf} = 2.5 f_\theta B \quad \cdots$$
$$U_{rf} = 1.2 E_{r1} U_d$$

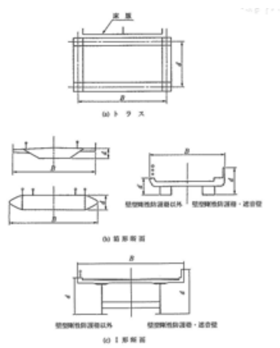
Based on wind tunnel test

smooth flow : $U_{rf} = 1.2 E_{r1} U_d$

full model in turbulent flow : $U_{rf} = 1.2 U_d$

U_{cf} : Critical wind speed (m/s)
f_θ : Natural freq. (1st torsional) (Hz)
B : Bridge deck width (m)
d : Bridge deck height (m)
U_{rf} : Examining wind speed (m/s)
E_{r1} : Correction factor due to wind fluctuation
U_d : Basic design wind speed (m/s)

Surface roughness category	E_{r1}
0	1.10
I	1.10
II	1.15
III	1.20
IV	1.25



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- flutter

$$U_{cf} > U_{rf}$$
$$U_{cf} = 2.5 f_\theta B \cdots$$
$$U_{rf} = 1.2 E_{r1} U_d$$

← for suspension br. and cable-stayed br. with box girder, or truss girder

for steel I-shaped girder br.

$$U_{cf} = \begin{cases} (6.0 - B/d) \cdot f_\theta \cdot B & (1.9 \le B/d < 3.5) \\ 2.5 \cdot f_\theta \cdot B & (3.5 \le B/d < 4.5) \\ 2.1 \cdot f_\theta \cdot B & (3.5 \le B/d < 4.5) \end{cases}$$

← And to be constructed above sea in low turbulence intensity

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- flutter

$$U_{cf} > U_{rf}$$
$$U_{cf} = 2.5 f_\theta B \cdots$$
$$U_{rf} = 1.2 E_{r1} U_d$$

If stability against flutter is examined by wind tunnel test,

- In case of section model test in uniform flow:
  the test should be done under the condition of incidence angle of wind,
  $-3\ (\text{deg}) \le \alpha \le +3\ (\text{deg})$

- In case of full bridge model in turbulent boundary layer in which the wind condition on bridge site is simulated:
  $U_{rf} = 1.2 U_d$

Bridge type	Susp. Truss g.	Susp. Box g.	Cable-st. Truss g.	Cable-st. Box g.	Girder St. sup.	Girder Rub. Sup.
Log. Damp.	0.03	0.02	0.03	0.02	$\frac{0.75}{\sqrt{L}}$	$\frac{0.75}{\sqrt{L}}$

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- galloping

$$U_{cg} > U_{rg}$$
$$U_{cg} = 8 f_h B \quad \cdots$$
$$U_{rg} = 1.2 U_d$$

U_{cg} : Critical wind speed (m/s)
U_{rg} : Examining wind speed (m/s)
f_h : Natural freq. (heaving 1st) (Hz)

- VIV

$$U_{cvh} > U_{rvh} \quad or \quad h_c < h_a$$
$$U_{cv\theta} > U_{rv\theta} \quad or \quad \theta_c < \theta_a$$

U_{cvh} : Onset wind speed for heav. (m/s)
U_{rvh} : Examining wind speed for heav. (m/s)
h_c : Maximum amp. for heav. (m)
h_a : Allowable amp. for heav.(m)
$U_{cv\theta}$: Onset wind speed for tors. (m/s)
$U_{rv\theta}$: Examining wind speed for tors. (m/s)
θ_c : Maximum amp. for tors. (deg.)
θ_a : Allowable amp. for tors.(deg.)

31

- galloping

$$U_{cg} > U_{rg}$$
$$U_{cg} = 8 f_h B \quad \cdots$$
$$U_{rg} = 1.2 U_d$$

← for suspension br. and cable-stayed br. with box girder, or truss girder and to be constructed above flat terrain and expected almost horizontal wind such as above sea and with steel supporting member

$U_{cg} = 4.5 \cdot f_h \cdot B$ for suspension br. and cable-stayed br. with box girder, or truss girder and to be constructed above flat terrain and expected almost horizontal wind such as above sea and with rubber supporting member

$U_{cg} = 4 \cdot f_h \cdot B$ for suspension br. and cable-stayed br. with box girder, or truss girder and to be constructed where a ridge, or a cape locates behind and is running in parallel to the bridge axis

$U_{cg} = 3.5 \cdot f_h \cdot B$ for steel I-shaped girder br.

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● VIV

$$U_{cvh} > U_{rvh} \quad or \quad h_c < h_a$$
$$U_{cv\theta} > U_{rv\theta} \quad or \quad \theta_c < \theta_a$$

U_{cvh} : Onset wind speed for heav. (m/s)
U_{rvh} : Examining wind speed for heav. (m/s)
h_c : Maximum amp. for heav. (m)
h_a : Allowable amp. for heav.(m)
$U_{cv\theta}$: Onset wind speed for tors. (m/s)
$U_{rv\theta}$: Examining wind speed for tors. (m/s)
θ_c : Maximum amp. for tors. (deg.)
θ_a : Allowable amp. for tors.(deg.)

$$U_{rvh} = U_d$$

For heaving response

● For suspension br. and cable-stayed br. with box girder

$$U_{cvh} = 2.0 \cdot f_h \cdot B$$

● For steel I-shaped girder

$$U_{cvh} = 2.0 \cdot f_h \cdot B \qquad (1.9 \le B/d < 4.5)$$
$$h_c = h_e \cdot E_{ms} \cdot E_{th}$$
$$h_e = \frac{\beta_h}{m_r \delta_h} \cdot B$$

h_e : Heavinq VIV amplitude in smooth flow at vibration mode =1.0

β_h : Heaving VIV amplitude equivalence factor
For suspension br. and cable-stayed br. with box girder: $\beta_h = 0.05(B/d)^{-1} \cdot \beta_{ds}$

For steel I-shaped girder:
$\beta_h = 0.04 \qquad (1.9 \le B/d < 3.5)$
$\beta_h = 0.14(B/d)^{-1} \quad (3.5 \le B/d < 4.5)$

β_{ds} : Correction factor related with cross section geometry of box girder

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m_r : Non-dimensional mass

δ_h : Logarithmic structural damping decrement for heaving mode

E_{ms} : Correction factor related to vibration mode:

E_{th} : Reduction factor due to wind turbulence:

$$E_{th} = 1 - 15 \cdot \beta_t \cdot (B/d)^{1/2} \cdot I_u^2$$

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Recommendations for
loads on buildings
Chap.6 Wind loading
(2004, AIJ)

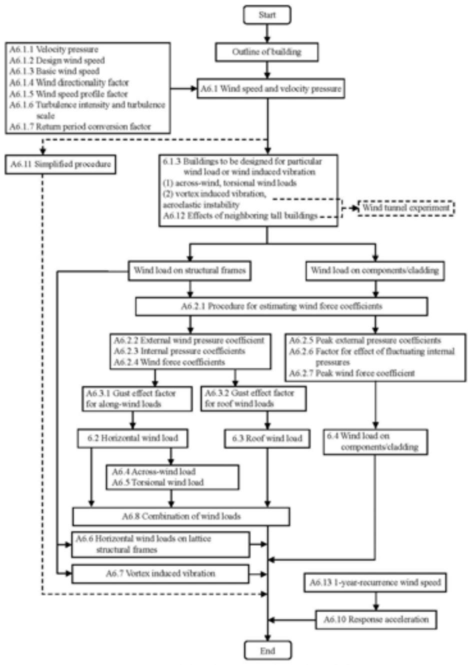

Figure 6.1.4 Flow chart for estimation of wind load

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Recommendations for
loads on buildings
Chap.6 Wind loading
(2004, AIJ)

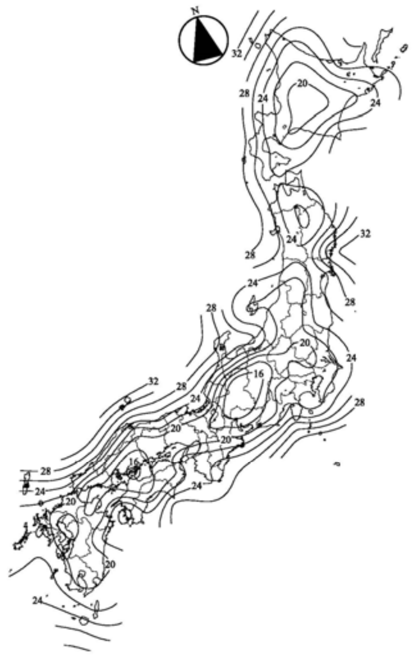

Figure A6.1.5 100-year-recurrence 10-minutes mean wind speed at 10m above ground over a flat and open terrain in winter (m/s)

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Appendix A Materials of Lectures and Seminars

A-7 Seminar for Cost Estimate

1

Seminar for Cost Estimate 2017/10/31 – 11/1

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Takuya KURAMOCHI
Cost Estimate in Bago River BridgeD/D
Nippon Koei

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Objective

To enhance understanding and importance of cost estimate from the view point of the employer

Contents

1. Cost Estimate
- Purpose/Meaning
- Japanese standard
- Myanmar standard

2. Cost Estimate Method

3. Cost Breakdown
Structure of Contract Amount in JP standard

4. Cost Breakdown

5. Exercise: PC-I girder (On-ramp of Bago Bridge)

*Trainee have start exercise from considering work items
- A_Superstructure
- B_Substructure
- C_Foundation

Infrastructure

▷ Characteristics

Infrastructure has important role to support citizens daily activity. Government has responsibility to build and maintain infrastructures.

Infrastructure characteristics are followings
- Financed by government (from citizens taxation)
- Only one in particular conditions
- Not only one way of construction to accomplish a project

Because of those characteristics, clear **justification(Fairness)** and **explanation** of project cost are required for public.

For this sake, specified cost estimate standards are important. In addition cost estimate standards dedicate to **keep infrastructure quality** and **continuous improvement of construction industry**.

About "Cost Estimate"

▷ Estimate an appropriate cost

All activities are converted into money

For employer,
To confirm expected project cost
*for bidding to have appropriate contract

For contractor,
To estimate bidding contract amount

Difference is whether a cost estimator should define project condition or not, as following 3 factor.

1_Preparing "Bidding Document"
2_Setting "Construction Conditions"
3_Setting "Cost Estimate Conditions"

These are Employers obligations

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▷ 2 steps of "Cost Estimate" as an employer

1st Step,
Definition of project condition

2nd Step
Estimation of "Contract amount", especially "construction cost" composed by required resources

*Contractors consider only this step. Project conditions are provided in a contract doc.

▷ Purpose of Cost Estimate

For Public

- Fairness/Justification / Clear Explanation
- Minimization of Cost
- Quality Control

= infrastructure should be commenced "Fairy / with reasonable price / better quality"

⇒Ordinary "**Trade off relation ship**"

⇒Fair cost estimate is necessary for everyone, employer and contractor, public

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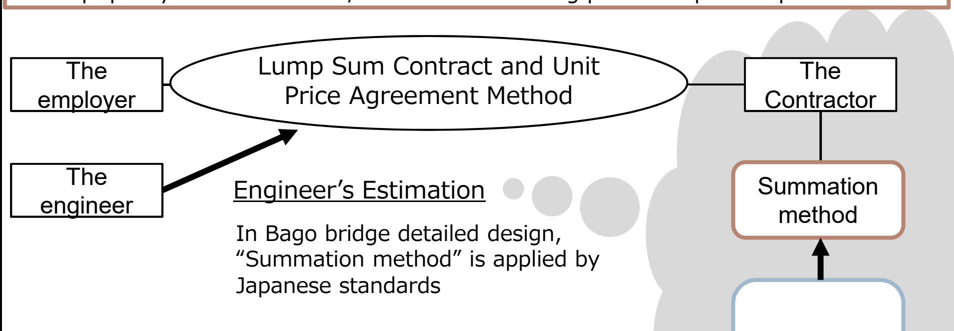
▷ International Bidding

"Lump Sum Contract and Unit Price Agreement Method".

An international bidding based on FIDIC pink book applies this contract

For international bidding, estimated cost is defined as "Engineer Estimation(Engineer cost)" In FIDIC.

"Engineer Estimation" is reference cost of bidding document.
To keep quality of infrastructure, evaluation of bidding price is important procedure.



*Pink book：
Multilateral Development Bank(MDB) Harmonised Edition
June 2010

Japanese Standard, Guideline, Manual

▷ Applied Japanese Standards for Bago Bridge DD

Japan International Cooperation Agency (JICA)

- Preparatory Survey for Grant Aid / Design and Cost Estimate Manual (Civil Engineering), 2016
- Preparatory Survey for Grant Aid / Design and Cost Estimate Manual (Equipment Procurement), 2016
- Preliminary Project Cost Estimate Guideline, 2008

Construction Research Institute

- Cost Estimate Standard for Civil Works (MLIT), 2016, 2013, 2012, 2011
- Cost Estimate Standard for Civil Works (MOC), 2000, 1999, 1993, 1992
- Cost Estimate Standard for Port Civil Works (MLIT), 2016
- Standard Specification of Cost Estimate for Civil Works (MLIT), 2016
- Construction Material Cost, April, 2017

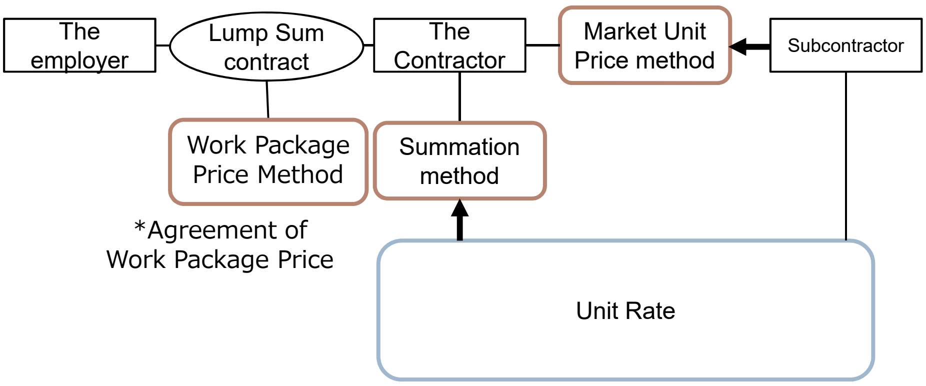
Japan Construction Machinery and Construction Association

- Construction Equipment Depreciation Calculation, 2016
- Bridge Erection Works Cost Estimation, 2016

Several Methods of Cost Estimate in Japan

1_Summation Method
2_Market Unit Price Method
3_Work Package Price Method
*appropriate English name is not sure

▷ Position of Every Method



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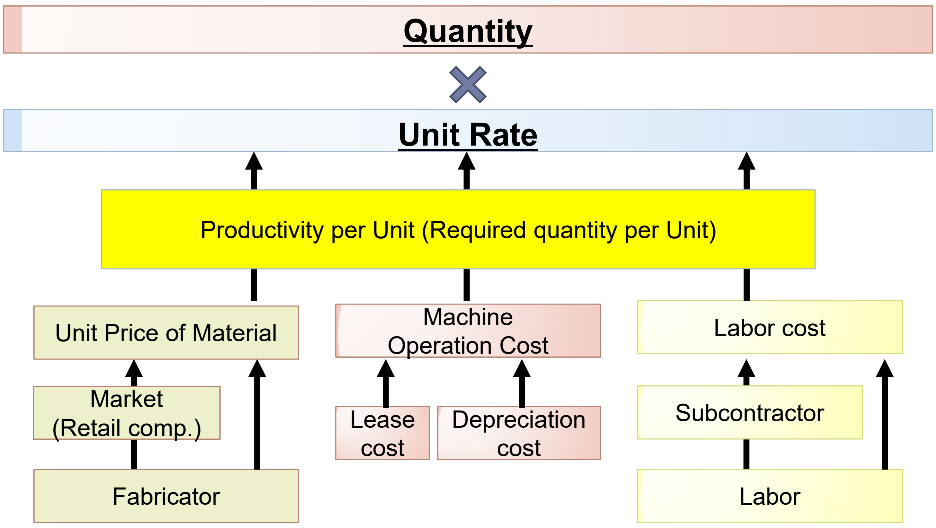
▷ 1_Summation Method

Conventional method in Japan
since when cost estimate started.



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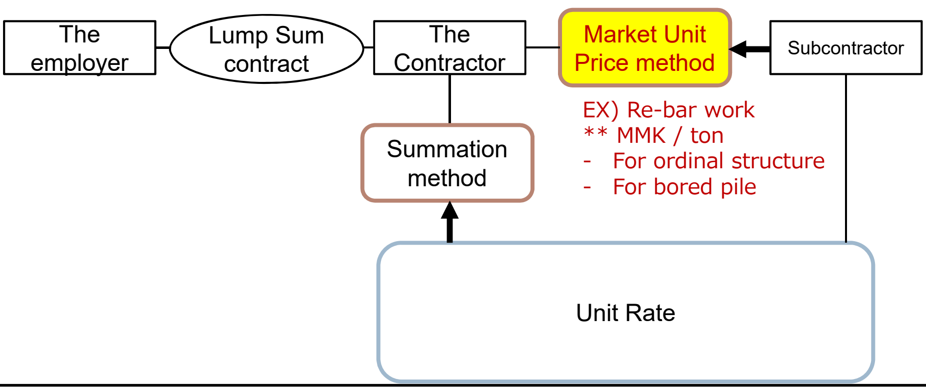
▷ 2_Market Unit Price Method

Work items have been outsourcing to subcontractor.

So an **unit price** is determined according to contract amount between “Contractor and Subcontractor” for **rationalization** of cost estimate.

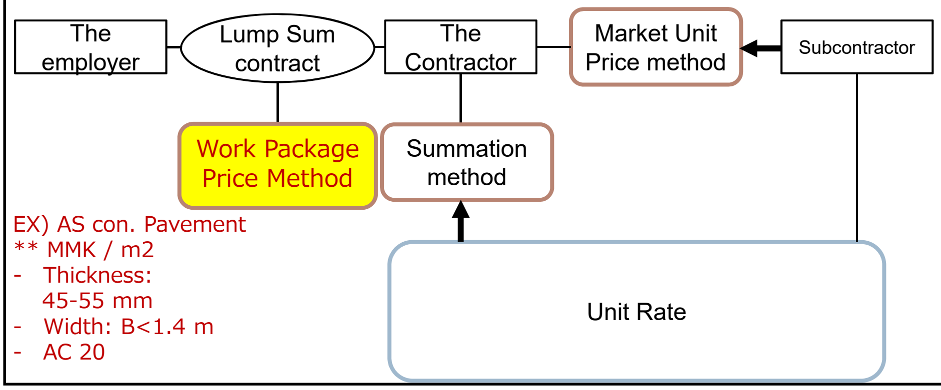
Those market prices are replaced in stead of unit prices by summation method



▷ 3_Work Package Price Method

This method is applied for "Lump Sum Contract and Unit Price Agreement Method".
*An international bidding based on FIDIC pink book applies this contract

A **package price** is already continuously published by government for **rationalization** and **justification** of cost estimate.
Focusing on structure, like AS con. pavement, concrete pier,,,



EX) AS con. Pavement
** MMK / m2
- Thickness: 45-55 mm
- Width: B<1.4 m
- AC 20

▷ Ex) 1,000m2 form work

Procedure / Method	1_Estimation of Work productivity (per day)	2_Work productivity (per day)	3_Setting Unit Rate	4_Calculation of Cost
0_Summation Method (from estimation of work productivity) Condition: 1,000m2 should be done in (33d)	Form worker: **3 person/30m2** Unskilled labor: **1 person/30m2**	Form worker: **0.1 person/m2** Unskilled labor: **0.03person/m2**	@20,000MMK/person*0.1 person/m2 =2,000MMK/m2 @13,000MMK/person*0.033 person/m2 =429 MMK/m2	(2,000+429) MMK/m2*1,000m2= 2,429,000 MMK
1_Summation Method		Form worker: **0.1 person/m2** Unskilled labor: **0.03 person/m2**	@20,000MMK/person*0.1 person/m2 =2,000MMK/m2 @13,000MMK/person*0.033 person/m2 =429 MMK/m2	(2,000+429) MMK/m2*1,000m2= 2,429,000 MMK
2_Market Unit Price Method			Form work unit rate: **2,000MMK/m2+500 MMK/m2 = 2,500 MMK/m2**	@2,500MMK * 1,000m2 = 2,500,000 MMK
3_Work Package Price Method				Form work 2,500/m2*1,000m2 = 2,500,000 MMK

Myanmar Cost Estimate Standard

▷ Standard

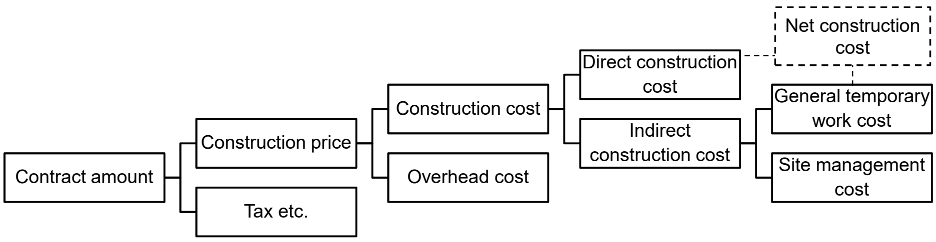
"Standard Data for working out Rates Per Unit Quantity of Items of Work-contd. Book"
" Material & Labour Price Book"

*Like "Market Unit Price Method" in Japan(in my understanding)

▷ Indirect Construction Cost

- Miscellaneous Amount
Maximum 10% of Total Material and Labour Amount
- Transportation Amount
Maximum 5% of Total Material Amount

Structure of Contract Amount in JP standard



- Physical contingency
- Price escalation
- Consultant fee
- Land acquisition etc. (The employer obligation)
are out of this structure

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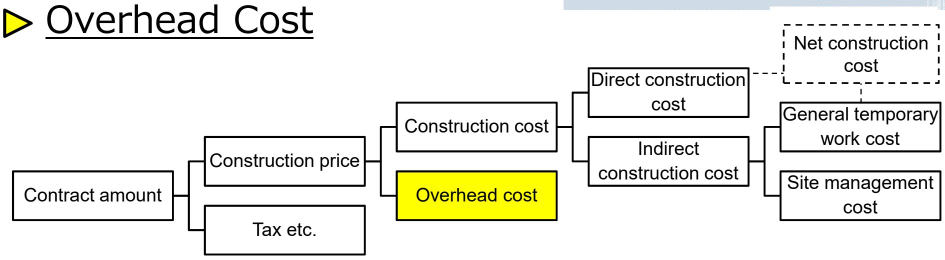
Miscellaneous Amount (maximum 10% of Total Material and Labour Amount and Transportation Amount (maximum 5% of Total Material Amount) 15

Overhead Cost



"Overhead Cost" is cost for
- Contractor's head office expense
- Cost for experiment to improve their technology
- Appropriate profit as public work

In Japanese standards, "Overhead Cost" is calculated as rate calculation depending on contract amount of "Construction cost"

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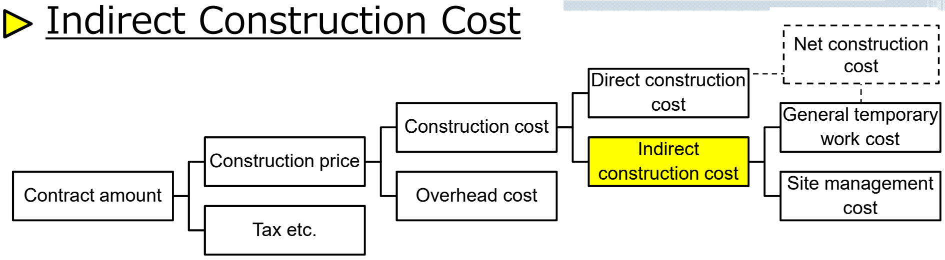
Miscellaneous Amount (maximum 10% of Total Material and Labour Amount and Transportation Amount (maximum 5% of Total Material Amount) 16

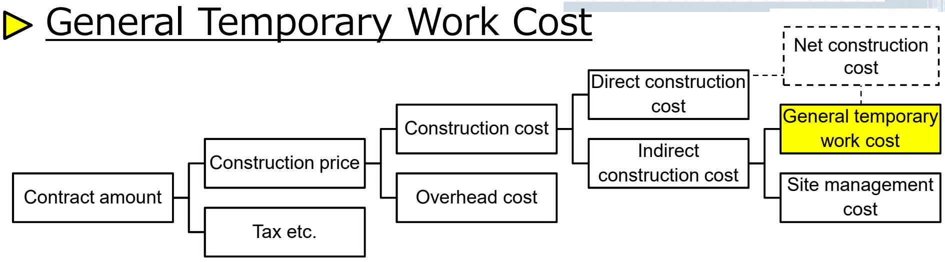
Indirect Construction Cost



"Indirect Construction Cost" is cost for "not particular object". In other word, general cost shared by all work items of direct construction cost.

Mainly Indirect construction cost is classified into 2 parts.
- General temporary work cost
- Site management cost

▷ General Temporary Work Cost

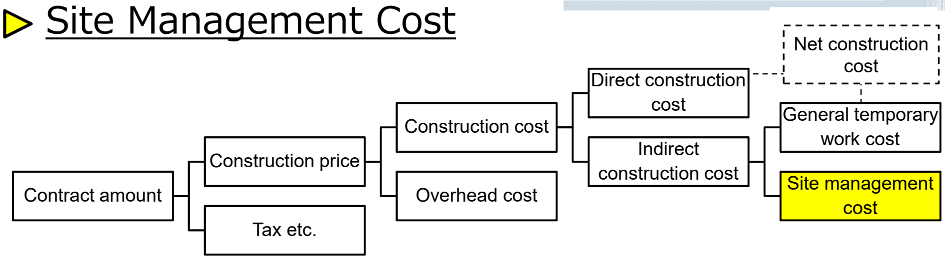


"General temporary work cost" is cost for "General works for all work items of direct construction cost".

EX) temporary road, temporary jetty, and management of site office etc. are for any kind of works in site

In Japanese standards, "General temporary work cost" is composed by **2 portions** "Rate calculation portion" and "Summation portion" Rate calculation portion is determined by rate and correction coefficient depending on "**Type of civil work**" and "**Amount of direct construction cost**"

▷ Site Management Cost



"Site management cost" is cost for "Management of site".

In Japanese standards, "Site management cost" is calculated as rate calculation.

Ratio and Correction coefficient in calculation are depending on "**Amount of net construction cost**"

Procedure of Cost Estimate

Construction Plan	Cost Estimate	Design / Document
	Composition / Breakdown Structure of work items (Category → Item → Specification)	
Construction plan	↓ Making of “Unit Rate” table for every work item	Bidding Document (Drawings / Specification)
↓ -Work schedule -Temporary work (plan and quantity) -Machine spec → input	↓ Direct Construction, Indirect Construction Cost, Overhead Cost	↓ input ← Quantity / Specification

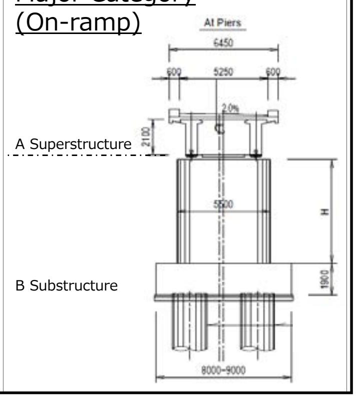
Exercise 1

▷ Breakdown Structure of Direct Construction cost

Let’s try making breakdown structure of direct construction cost of “PC-I girder”!!

In Bago bridge DD, PC-I girder is applied for On-ramp and F.O. in PKG3

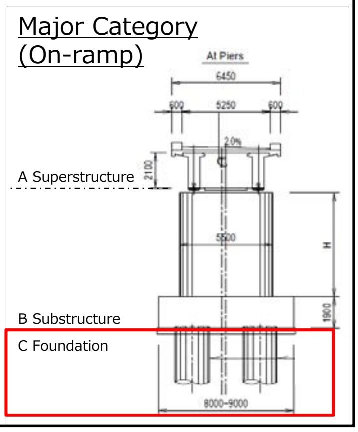
Major Category (On-ramp)



Exercise 2

▷ Making of “Unit Rate” table for “Foundation”

Let’s try making Unit Rate table of “Foundation”!!



▷ Expense of Equipment

Mainly 2 ways

1_Lease cost

1.1_All equipment except for cranes

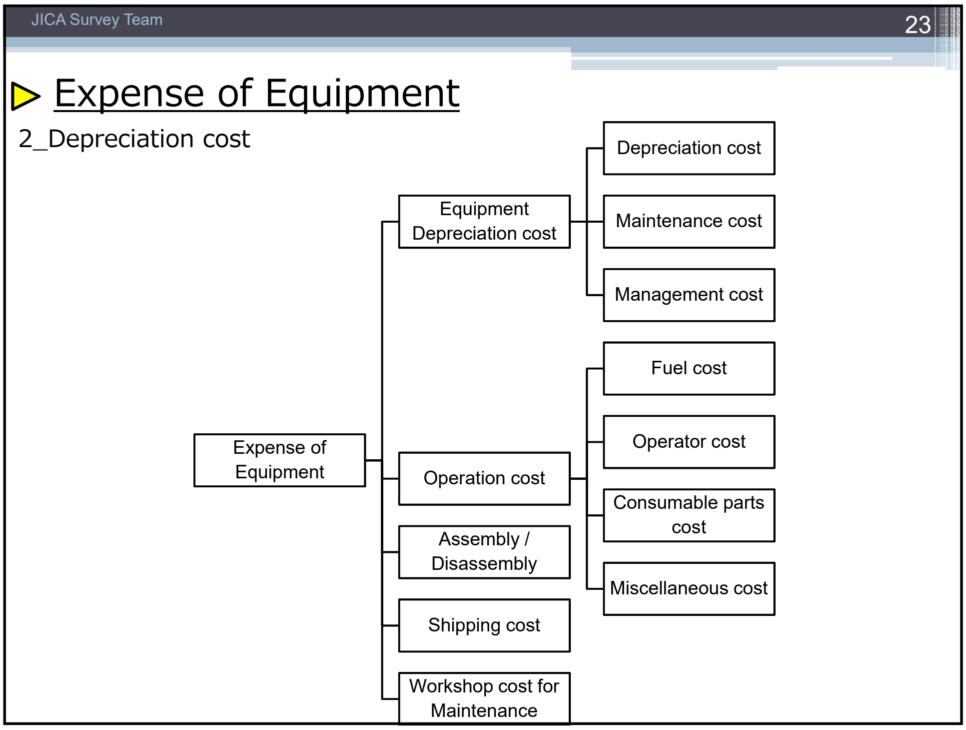
Lease cost = Market price * lease days

1.2_Cranes

Lease cost = Market price * operation days / months

2_Depreciation cost

Next page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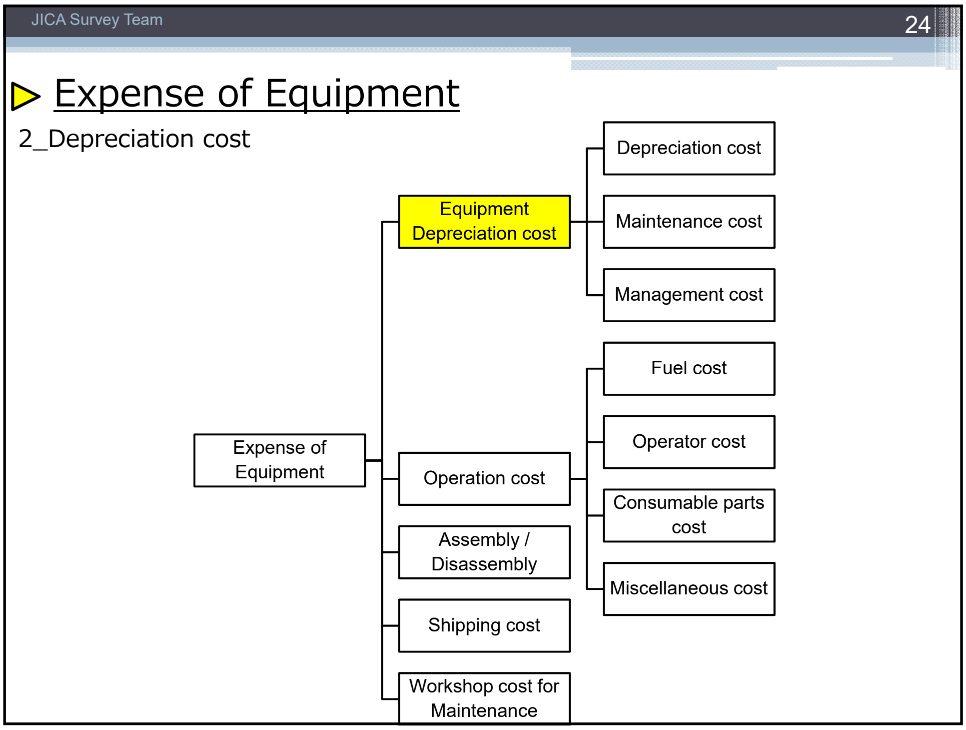
Expense of Equipment
2_Depreciation cost
Expense of Equipment
Equipment Depreciation cost
Depreciation cost
Maintenance cost
Management cost
Operation cost
Fuel cost
Operator cost
Consumable parts cost
Miscellaneous cost
Assembly / Disassembly
Shipping cost
Workshop cost for Maintenance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Expense of Equipment
2_Depreciation cost
Expense of Equipment
Equipment Depreciation cost
Depreciation cost
Maintenance cost
Management cost
Operation cost
Fuel cost
Operator cost
Consumable parts cost
Miscellaneous cost
Assembly / Disassembly
Shipping cost
Workshop cost for Maintenance

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▷ Equipment Depreciation cost

OPE- 2 Backhoe Crawler type Full bucket0.5m3 Source:

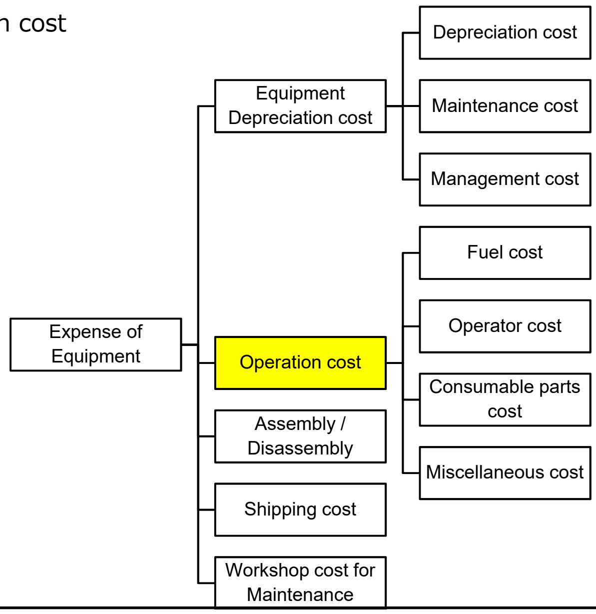
No.	Item Conditions	Category	Item	Specification	Unit	Quantity	Unit Price		Subtotal		Total
							Local(MMK)	Foreign(JPY)	Local(MMK)	Foreign(JPY)	Yen conv.
	OPE- 2		Backhoe Crawler type Full bucket0.5		day	1			316,684		26,348
	OPE- 2		Backhoe Crawler type Full bucket0.5		day	1	316,684		316,684		26,348
No.	Item Conditions	Category	Item	Specification	Unit	Quantity	Unit Price		Subtotal		Total
							Local(MMK)	Foreign(JPY)	Local(MMK)	Foreign(JPY)	Yen conv.
	Lab-12		Equipment Operator		person	1	18,270		18,270		1,520
			Machine_depreciation / rental_cost		day	1.28	200,000		256,000		21,299
			Diesel		L	61.47	690		42,414		3,529

Note
- Construction Equipment Rental Table/2016 P02-7
  Standard Operating Hours Per Year: a = 690 h / Standard Operating Days Per Year: b = 110 days
  Fuel Consumption Per Hour: c = 9.8 ℓ/h
  Fuel Consumption Per Day: d = a / b×c = 690/110×9.8 = 61.47 ℓ
- Equipment Operation Unit Cost Per Hour
  Hours Operating Per Day = a / b = 690/110 = 6.27 h

JICA Survey Team 26

▷ Expense of Equipment

2_Depreciation cost



▷ Operation Cost

OPE- 2 Backhoe Crawler type Full bucket0.5m3 Source:

No.	Item Conditions	Category	Item	Specification	Unit	Quantity	Unit Price Local(MMK)	Unit Price Foreign(JPY)	Subtotal Local(MMK)	Subtotal Foreign(JPY)	Total Yen conv.
	OPE- 2		Backhoe Crawler type Full bucket0.5		day	1			316,684		26,348
	OPE- 2		Backhoe Crawler type Full bucket0.5		day	1	316,684		316,684		26,348
No.	Item Conditions	Category	Item	Specification	Unit	Quantity	Unit Price Local(MMK)	Unit Price Foreign(JPY)	Subtotal Local(MMK)	Subtotal Foreign(JPY)	Total Yen conv.
	Lab-12		Equipment Operator		person	1	18,270		18,270		1,520
			Machine_depreciation / rental_cost		day	1.28	200,000		256,000		21,299
			Diesel		L	61.47	690		42,414		3,529

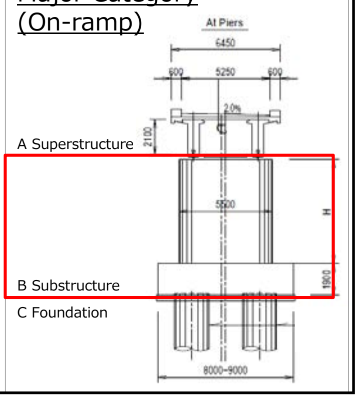
Note
- Construction Equipment Rental Table/2016 P02-7
  Standard Operating Hours Per Year: a = 690 h / Standard Operating Days Per Year: b = 110 days
  Fuel Consumption Per Hour: c = 9.8 ℓ/h
  Fuel Consumption Per Day: d = a / b×c = 690/110×9.8 = 61.47 ℓ
- Equipment Operation Unit Cost Per Hour
  Hours Operating Per Day = a / b = 690/110 = 6.27 h

Exercise 3

▷ Making of "Unit Rate" table for "Substructure"

Let's try making Unit Rate table of "Substructure"!!

Major Category (On-ramp)



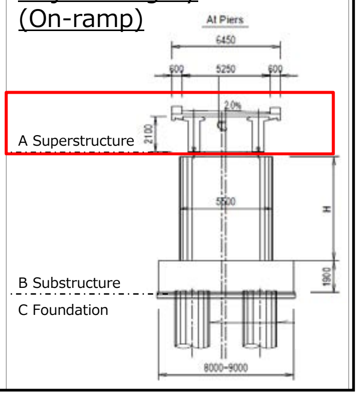
JICA Survey Team 29

Exercise 4

▷ Making of "Unit Rate" table for "Superstructure"

Let's try making Unit Rate table of "Superstructure"!!

Major Category
(On-ramp)



Appendix A Materials of Lectures and Seminars

A-8 Detailed Practice for Steel Cable Stayed Bridge

Date			No.	Contents	
Jan.	1st	Mon			
	2nd	Tue			
	3rd	Wed			
	4th	Thu			
	5th	Fri			
	6th	Sat			
	7th	Sun			
	8th	Mon	1	Introduction	CSB at Bago Bridge
	9th	Tue	2	Modeling	Material Properties
	10th	Wed	3		Section Properties
	11th	Thu	4		Design Load
	12th	Fri	5		Boundary Conditions
	13th	Sat			
	14th	Sun			
	15th	Mon	6	Parametric Analysis	Cable Pre-stress
	16th	Tue	7		Cable Number
	17th	Wed	8		Cable Arrangement
	18th	Thu	9		Girder Height
	19th	Fri	10		Tower Height
	20th	Sat			
	21st	Sun			
	22nd	Mon	11	Section Calculation	Section Calculation Sheet 1
	23rd	Tue	12		Section Calculation Sheet 2
	24th	Wed	13		Section Calculation Sheet 3
	25th	Thu	14		Design Check by JSHB 1
	26th	Fri	15		Design Check by JSHB 2
	27th	Sat			
	28th	Sun			
	29th	Mon	16		Design Check by JSHB 3
	30th	Tue	17		Design Check by JSHB 4
	31st	Wed	18		Design Check by JSHB 5

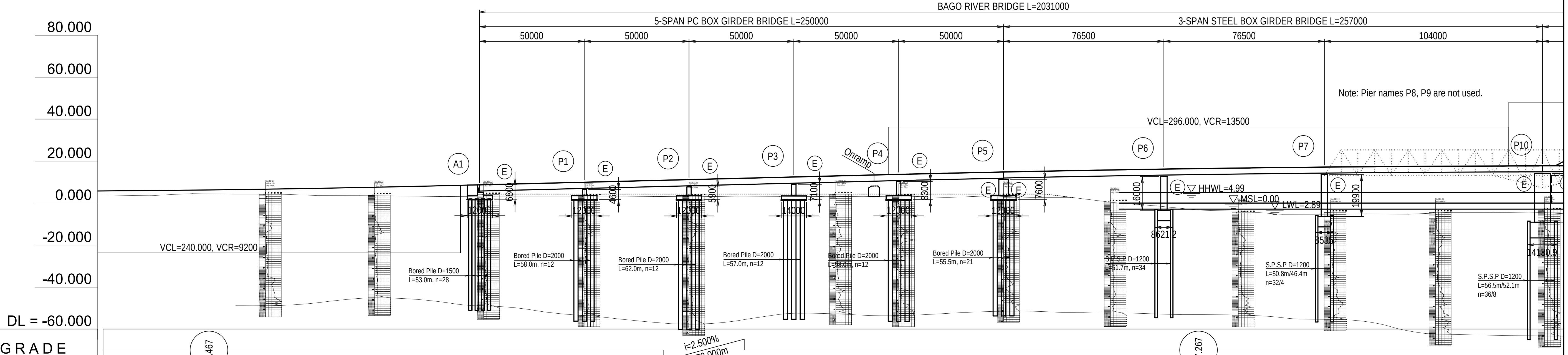
Date			No.	Contents	
Feb.	1st	Thu	19	Re-Analysis	Re-modeling
	2nd	Fri	20		Re-Calculation
	3rd	Sat			
	4th	Sun			
	5th	Mon	21		Re-Design Check 1
	6th	Tue	22		Re-Design Check 2
	7th	Wed	23		Re-Design Check 3
	8th	Thu	24	Seismic Design	Seismic Design by JSHB
	9th	Fri	25		Support Condition
	10th	Sat			
	11th	Sun			
	12th	Mon			
	13th	Tue	26	Design Exercise	Introduction
	14th	Wed	27		Modeling
	15th	Thu	28		Material, Section Properties
	16th	Fri	29		Load Conditions
	17th	Sat			
	18th	Sun			
	19th	Mon	30		Cable Pre-stress
	20th	Tue	31		Section Calculation 1
	21st	Wed	32		Re-analysis
	22nd	Thu	33		Section Calculation 2
	23rd	Fri	34	Presentation	Presentation Preparation 1
	24th	Sat			
	25th	Sun			
	26th	Mon	35		Presentation Preparation 2
	27th	Tue	36		Presentation Preparation 3
	28th	Wed	37	Closing	Presentation by Trainee

Extra Step	Construction Stage Analysis	Construction Step
		Modeling
		Analysis
		Design Check

GENERAL VIEW OF BAGO RIVER BRIDGE (4)

LONGITUDINAL SECTION

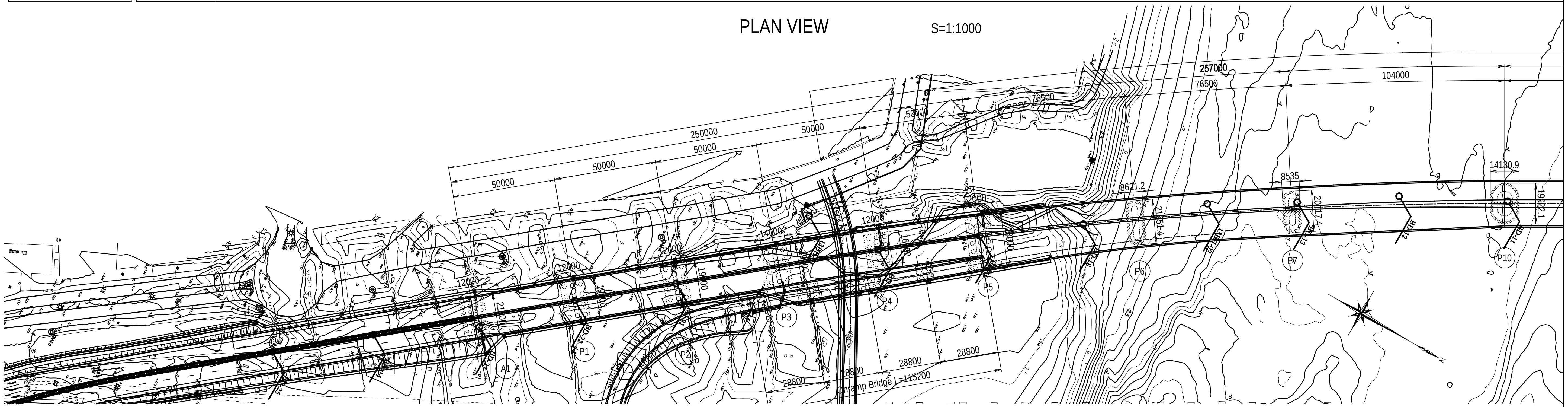
S=1:1000



GRADE	5.467	i=2.500% L=472.000m												17.267			
PROPOSED HEIGHT	5.953	6.247	7.392	8.692	9.767	9.942	11.192	12.267	12.442	13.691	14.681	14.830	16.211	16.453	17.159	17.481	17.757
GROUND HEIGHT	3.514	4.389	3.558	3.225	3.240	3.253	3.026	3.161	3.156	3.262	3.383	3.152	-1.721	-3.338	-5.346	-5.298	-4.549
CHAINAGE	0+200.000	0+228.000	0+300.000	A1 0+357.000	0+400.000	0+407.000 P1	P2 0+457.000	0+500.000	0+507.000 P3	P4 0+557.000	0+600.000	0+607.000 P5	P6 0+683.500	0+700.000	P7 0+760.000	0+800.000	P10 0+864.000
CURVE ELEMENT	R=∞ L=309.188									R=2000 L=335.621							
SUPER ELEVATION	i=2.000%	i=2.000%															

PLAN VIEW

S=1:1000

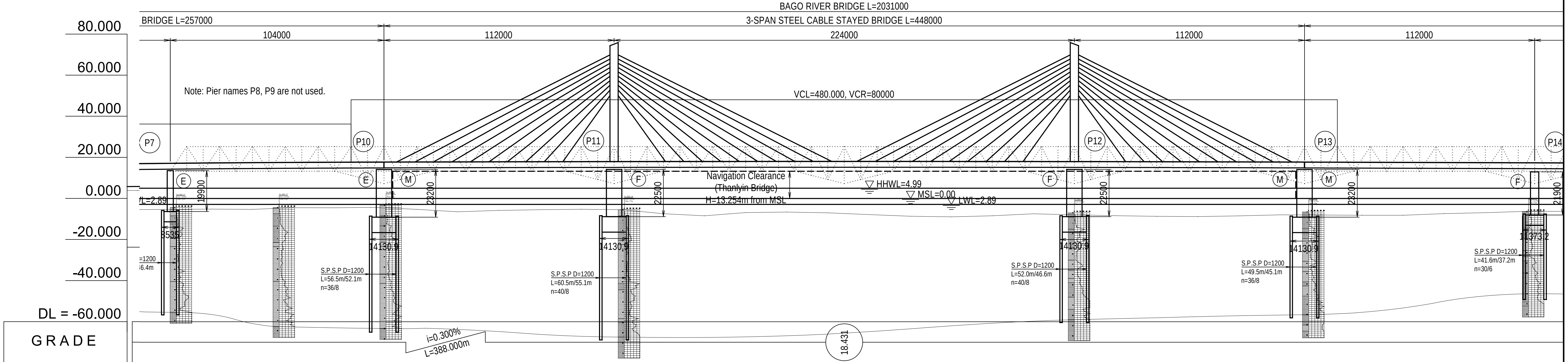


Elevation represents above MSL unless otherwise indicated.

PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T. HAYAKAWA		27 Nov. 2017	GENERAL VIEW OF BAGO RIVER BRIDGE (4)	1
				CHECKED BY	T. HAYAKAWA		28 Nov. 2017		DWG No.
				APPROVED BY	Y. SANO		29 Nov. 2017		P1-GE-0005

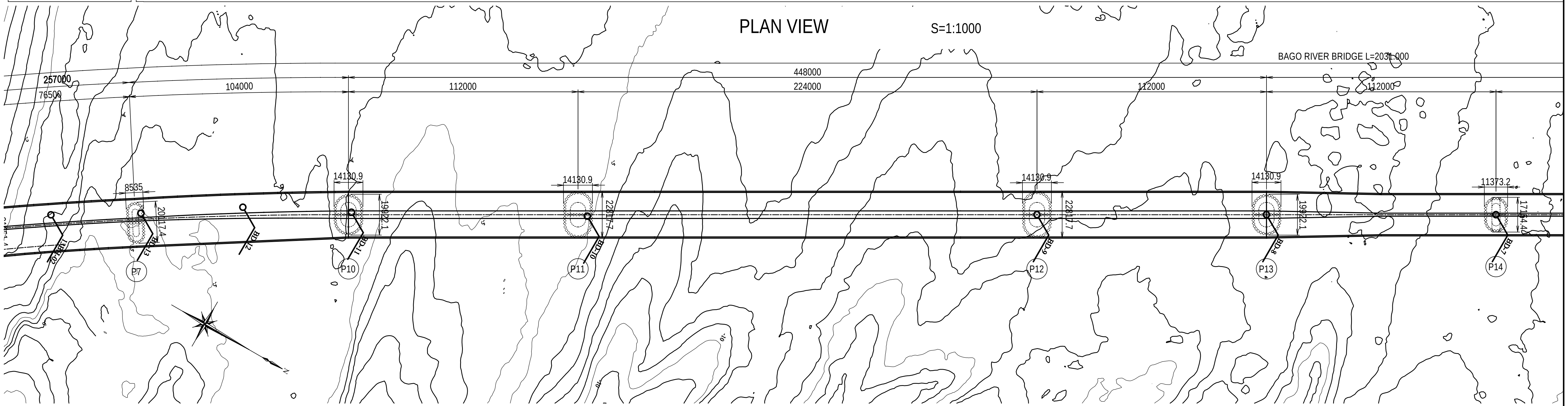
GENERAL VIEW OF BAGO RIVER BRIDGE (5)

LONGITUDINAL SECTION S=1:1000



GRADE	i=0.300% L=388.000m						18.431						
PROPOSED HEIGHT	17.159	17.481	17.757	17.850	17.993	18.023	18.071	18.070	17.993	17.790	17.757	17.495	17.423
GROUND HEIGHT	-5.346	-5.298	-4.549	-6.942	-5.411	-7.424	-7.329	-7.843	-7.959	-7.969	-8.024	-6.887	-6.276
CHAINAGE	P7 0+760.000	0+800.000	P10 0+864.000	0+900.000	P11 0+976.000	1+ 0.000	1+ 88.000	1+100.000	P12 1+200.000	1+300.000	1+312.000 P13	1+400.000	1+424.000 P14
CURVE ELEMENT	R=∞ L=1769.899												
SUPER ELEVATION	i=2.000%												

PLAN VIEW S=1:1000



Elevation represents above MSL unless otherwise indicated.

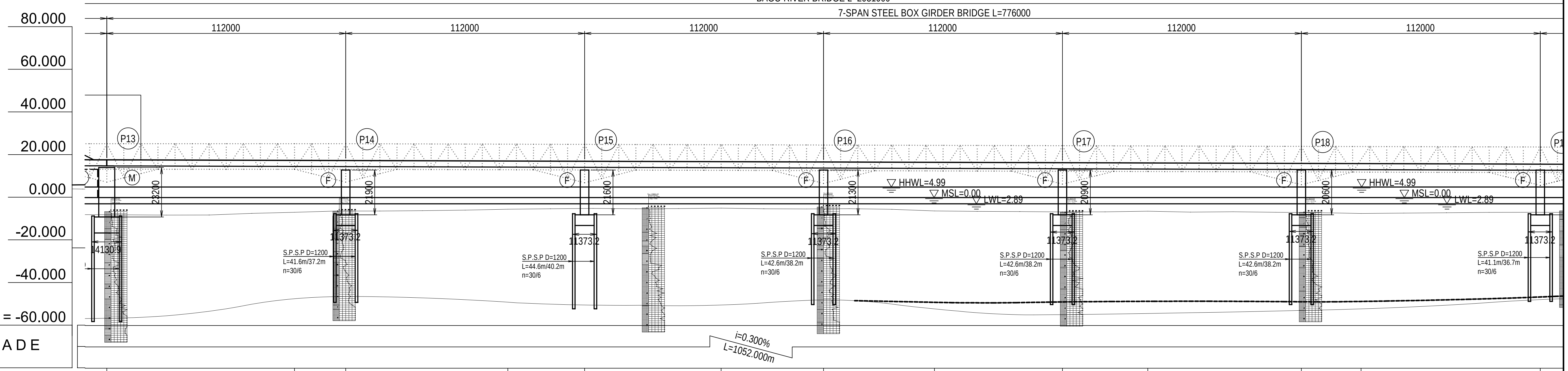
PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.

	NAME	SIGNATURE	DATE
PREPARED BY	T. HAYAKAWA	早川 知邦	27 Nov. 2017
CHECKED BY	T. HAYAKAWA	早川 知邦	28 Nov. 2017
APPROVED BY	Y. SANO	佐野 [illegible]	29 Nov. 2017

DRAWING TITLE	PACKAGE	DWG No.
GENERAL VIEW OF BAGO RIVER BRIDGE (5)	1	P1-GE-0006

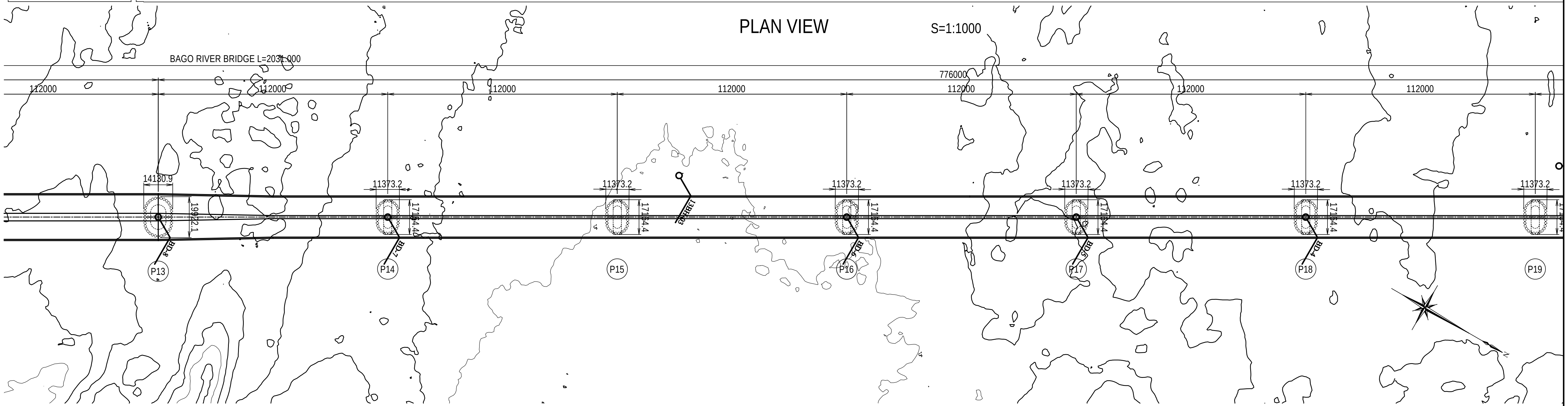
GENERAL VIEW OF BAGO RIVER BRIDGE (6)

LONGITUDINAL SECTION S=1:1000



GRADE	i=0.300% L=1052.000m												
PROPOSED HEIGHT	17.757	17.495	17.423	17.195	17.087	16.895	16.751	16.595	16.415	16.295	16.079	15.995	15.743
GROUND HEIGHT	-8.024	-6.887	-6.276	-5.462	-5.091	-4.926	-5.262	-6.249	-6.695	-6.392	-6.989	-7.426	-6.878
CHAINAGE	1+312.000 (P13)	1+400.000	1+424.000 (P14)	1+500.000	1+536.000 (P15)	1+600.000	1+648.000 (P16)	1+700.000	1+760.000 (P17)	1+800.000	1+872.000 (P18)	1+900.000	1+984.000 (P19)
CURVE ELEMENT	R=∞ L=1769.899												
SUPER ELEVATION	i=2.000%												

PLAN VIEW S=1:1000



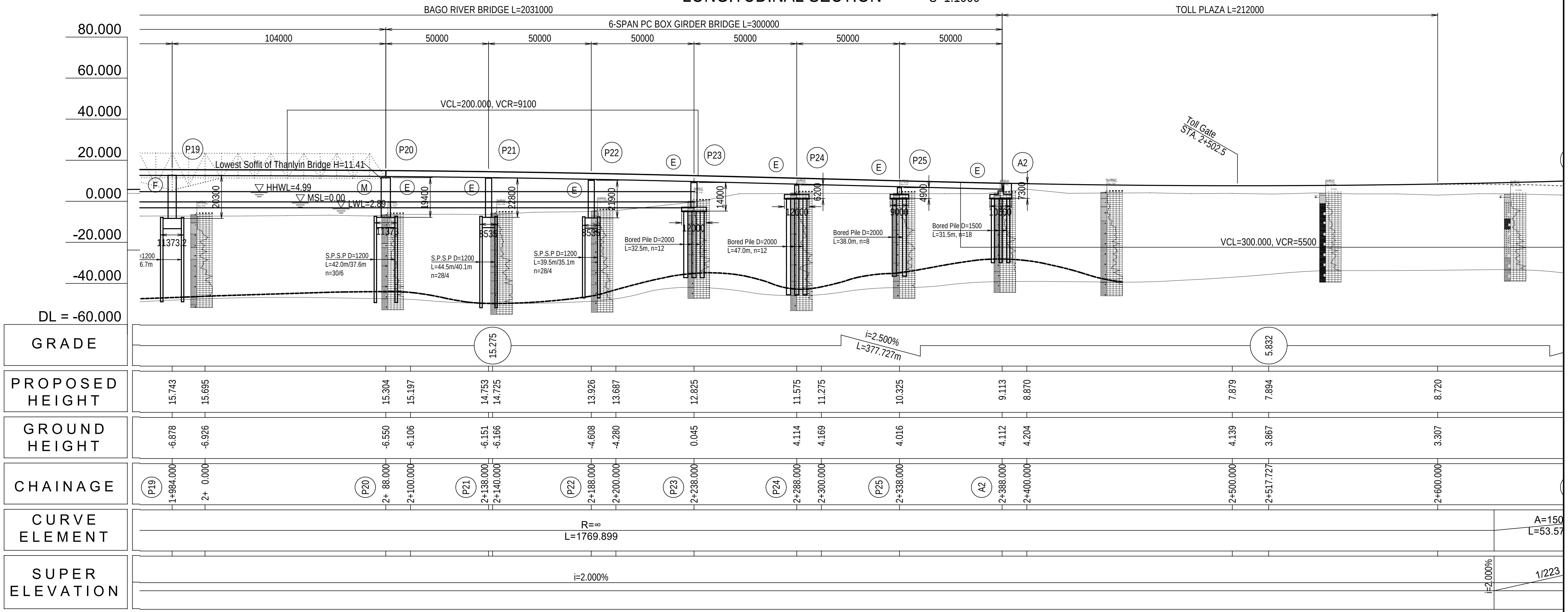
Elevation represents above MSL unless otherwise indicated.

PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T. HAYAKAWA		27 Nov. 2017	GENERAL VIEW OF BAGO RIVER BRIDGE (6)	1
				CHECKED BY	T. HAYAKAWA		28 Nov. 2017		DWG No.
				APPROVED BY	Y. SANO		29 Nov. 2017		P1-GE-0007

GENERAL VIEW OF BAGO RIVER BRDIGE (7)

LONGITUDINAL SECTION

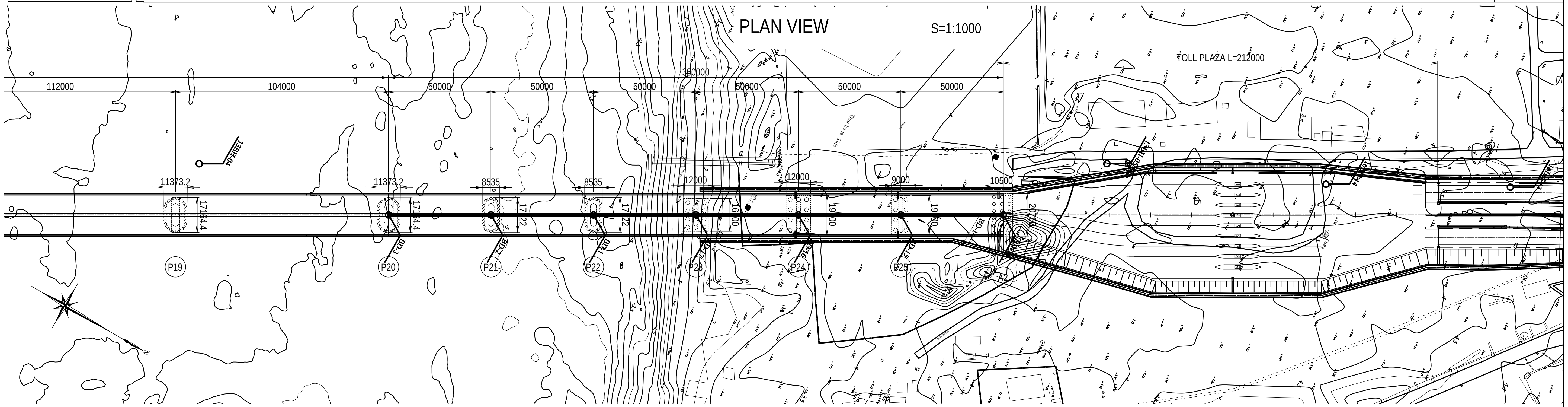
S=1:1000



PROPOSED HEIGHT	15.743	15.695	15.304	15.197	14.753	14.725	13.926	13.687	12.825	11.575	11.275	10.325	9.113	8.870	7.879	7.894	8.720		
GROUND HEIGHT	-6.878	-6.926	-6.550	-6.106	-6.151	-6.166	-4.608	-4.280	0.045	4.114	4.169	4.016	4.112	4.204	4.139	3.867	3.307		
CHAINAGE	1+984.000	2+ 0.000	2+ 88.000	2+100.000	2+138.000	2+140.000	2+188.000	2+200.000	2+238.000	2+288.000	2+300.000	2+338.000	2+388.000	2+400.000	2+500.000	2+517.727	2+600.000		

PLAN VIEW

S=1:1000

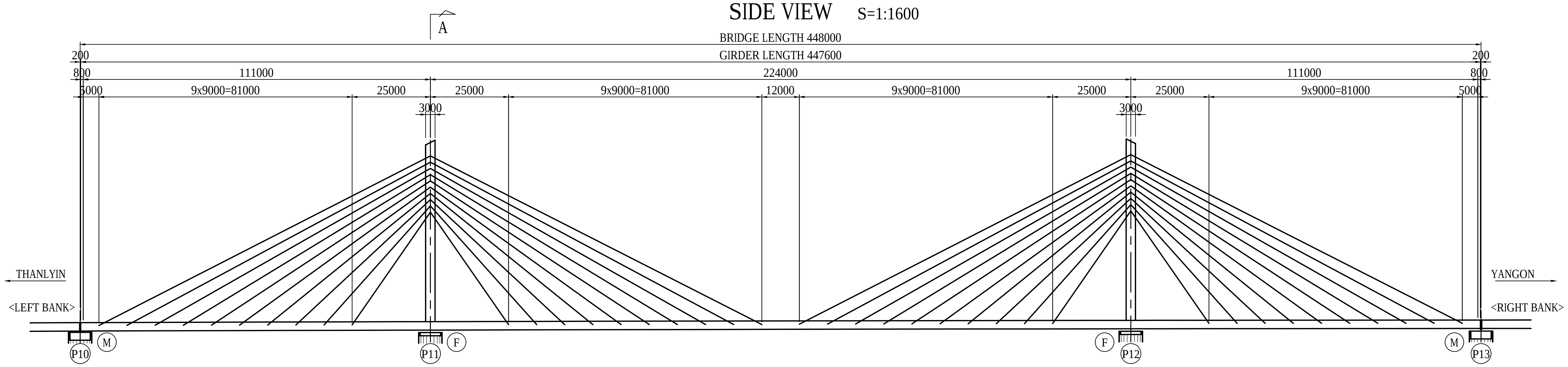


Elevation represents above MSL unless otherwise indicated.

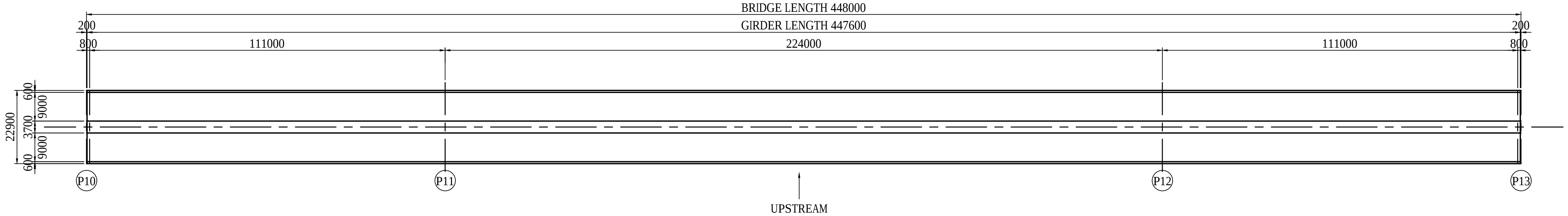
PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T. HAYAKAWA		27 Nov. 2017	GENERAL VIEW OF BAGO RIVER BRDIGE (7)	1
				CHECKED BY	T. HAYAKAWA		28 Nov. 2017		DWG No.
				APPROVED BY	Y. SANO		29 Nov. 2017		P1-GE-0008

GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (1)

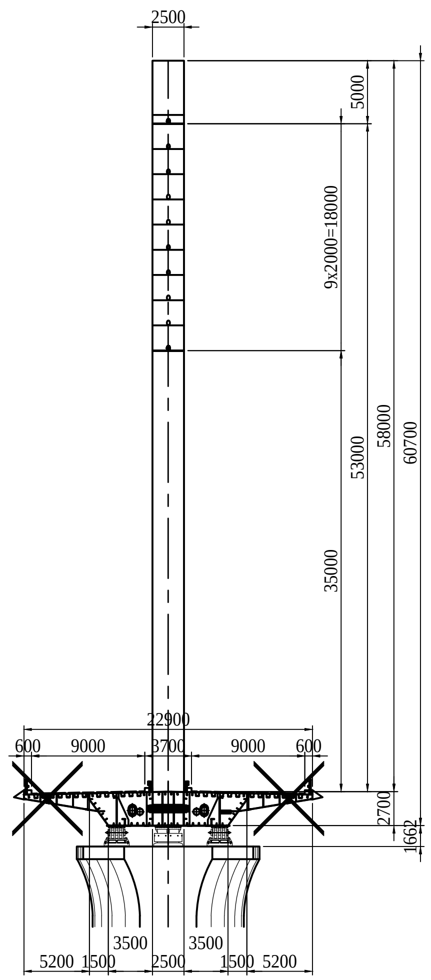
SIDE VIEW S=1:1600



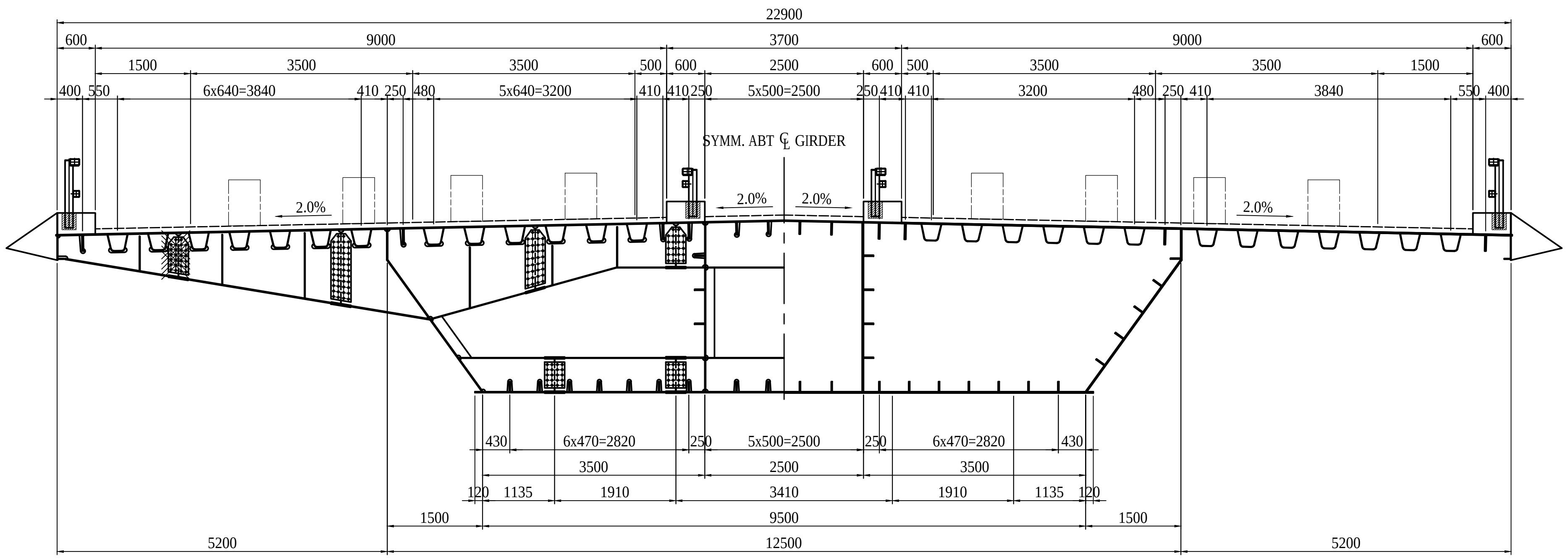
PLAN VIEW S=1:1600



CROSS-SECTION A-A S=1:600



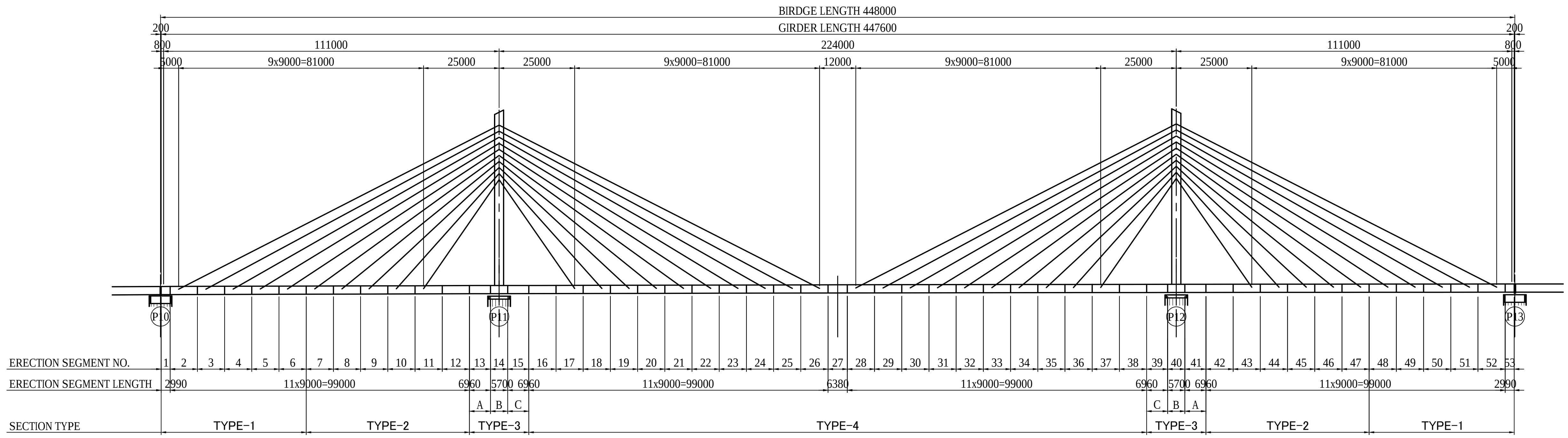
STANDARD HALF CROSS-SECTION S=1:100



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017	GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (1)	1
				CHECKED BY	T. HAYAKAWA	[signature]	28. Nov.2017		DWG No.
				APPROVED BY	Y. SANO	[signature]	29. Nov.2017		P1-CS-0001

GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (2)

SIDE VIEW



UNIT: mm

DECK PL	THK	16	16	16	16	16	16	16
	STEEL GRADE	SM400	SM400	a b a	SM400	a b a	SM400	SM400
	TROUGH	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8
	LONGIT. RIB A	250 x 24	250 x 24	250 x 24	250 x 24	250 x 24	250 x 24	250 x 24
	LONGIT. RIB B	200 x 20	200 x 20	200x20	200 x 20	200x20	200 x 20	200 x 20
OUTER WEB	THK	14	14	17	14	17	14	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
INNER WEB	THK	14	14	18 35 18	14	18 35 18	14	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16	200 x 20	160 x 16	160 x 16
BOTTOM FLANGE	THK	14	11	15	11	15	11	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16	200 x 20	160 x 16	160 x 16

a : SM400A
b : SM490Y

PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.

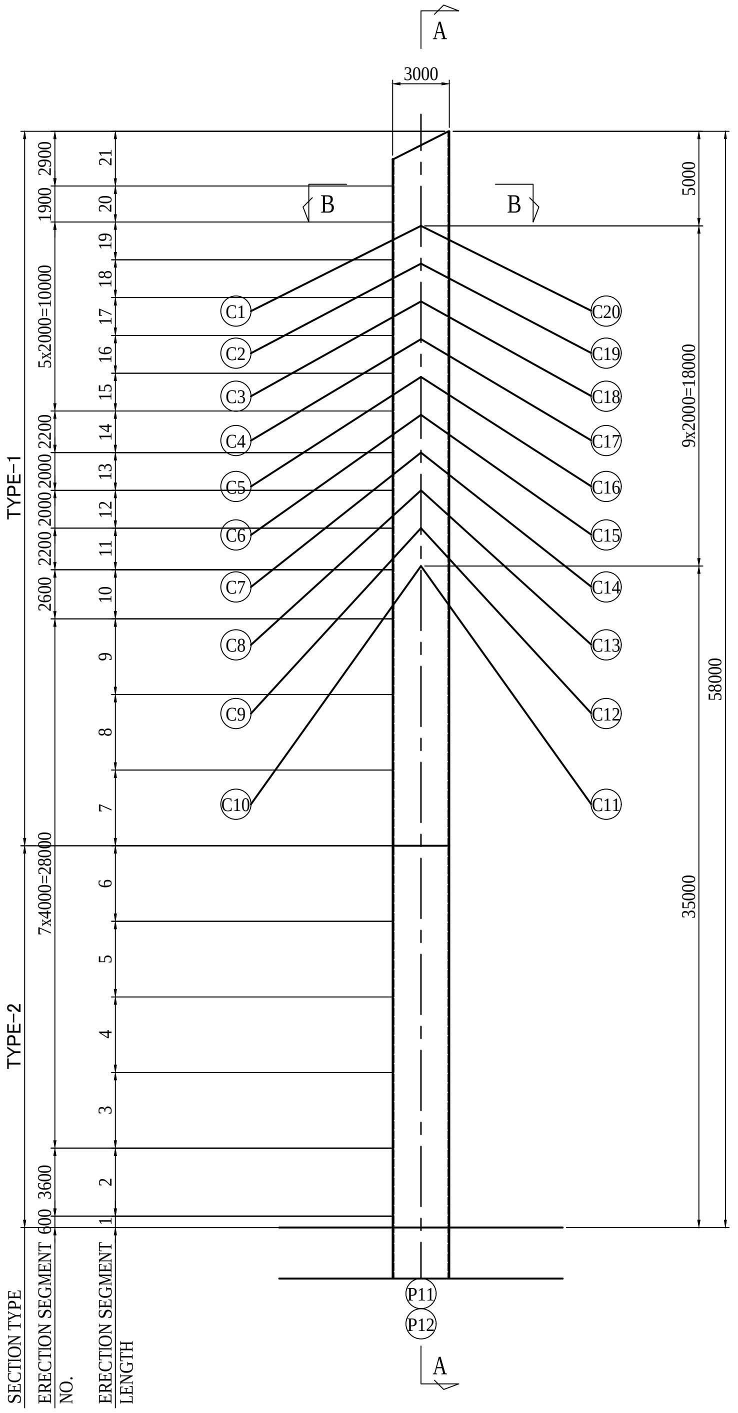
	NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017	GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (2)	1
CHECKED BY	T. HAYAKAWA	早川知邦	28. Nov.2017		DWG No.
APPROVED BY	Y. SANO	[illegible]	29. Nov.2017		P1-CS-0002

GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (3)

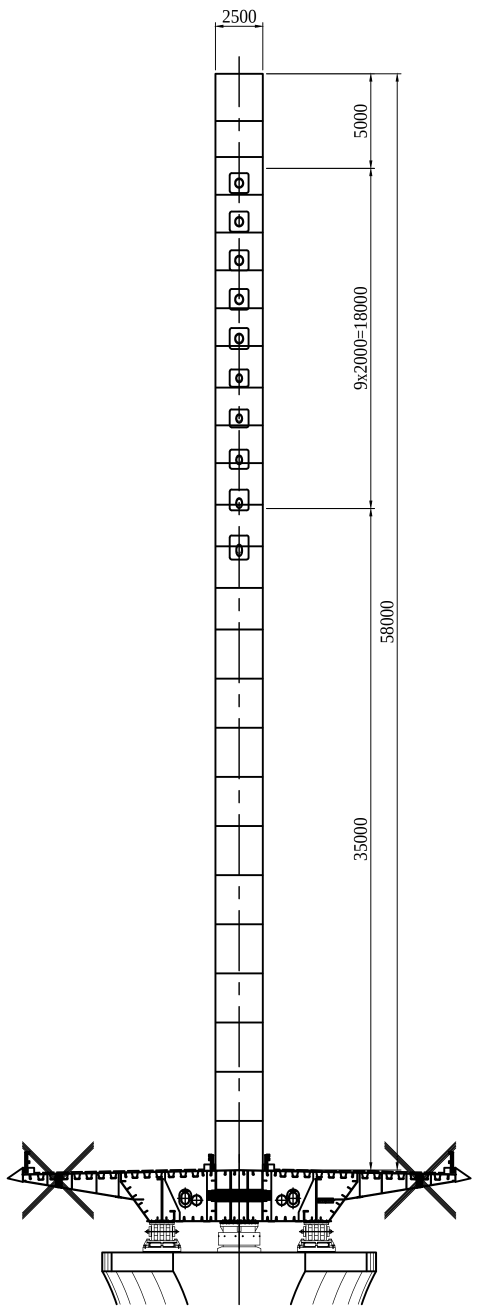
UNIT: mm

		TYPE-1	TYPE-2
FLANGE	STEEL GRADE	SM490Y	SM490Y
	THK	2420x40	2430x35
	LONGIT. RIB	2-280x27	2-260x25
WEB	STEEL GRADE	SM490Y	SM490Y
	THK	3000x40	3000x35
	LONGIT. RIB	5-250x25	5-250x25

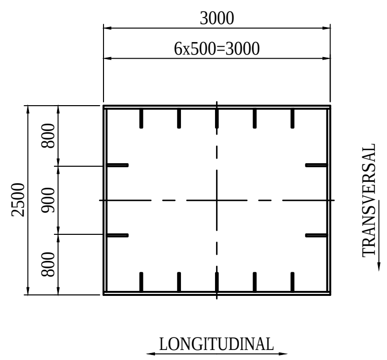
SIDE VIEW S=1:400



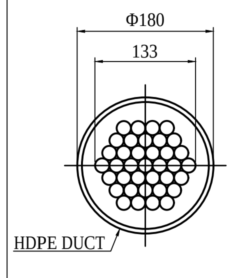
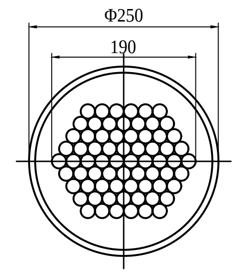
SECTION A-A



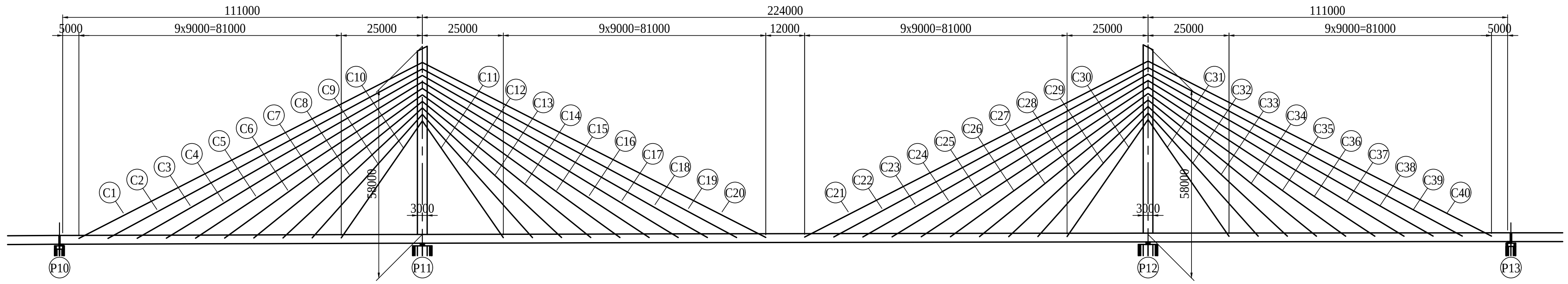
SECTION B-B



STAY CABLE NO.	NOS.	TYPE
C1	2	FUT-H (Φ15.6×70)
C2	2	
C3	2	
C4	2	
C5	2	
C6	2	FUT-H (Φ15.6×37)
C7	2	
C8	2	
C9	2	
C10	2	
C11	2	FUT-H (Φ15.6×37)
C12	2	
C13	2	
C14	2	
C15	2	
C16	2	FUT-H (Φ15.6×70)
C17	2	
C18	2	
C19	2	
C20	2	

ANCHORAGE TYPE	37H	70H
CABLE CROSS SECTION		
NOM.AREA	5420 mm^2	10255 mm^2
TENSILE STRENGTH	9657 kN	18270 kN
ELASTIC MODULUS	190 kN/mm^2	190 kN/mm^2
UNIT WEIGHT (STRANDS+HDPE DUCT)	50.8 kg/m (37×1.288+3.1)	96.0 kg/m (70×1.288+5.8)

LAYOUT DRAWING S=1:1600



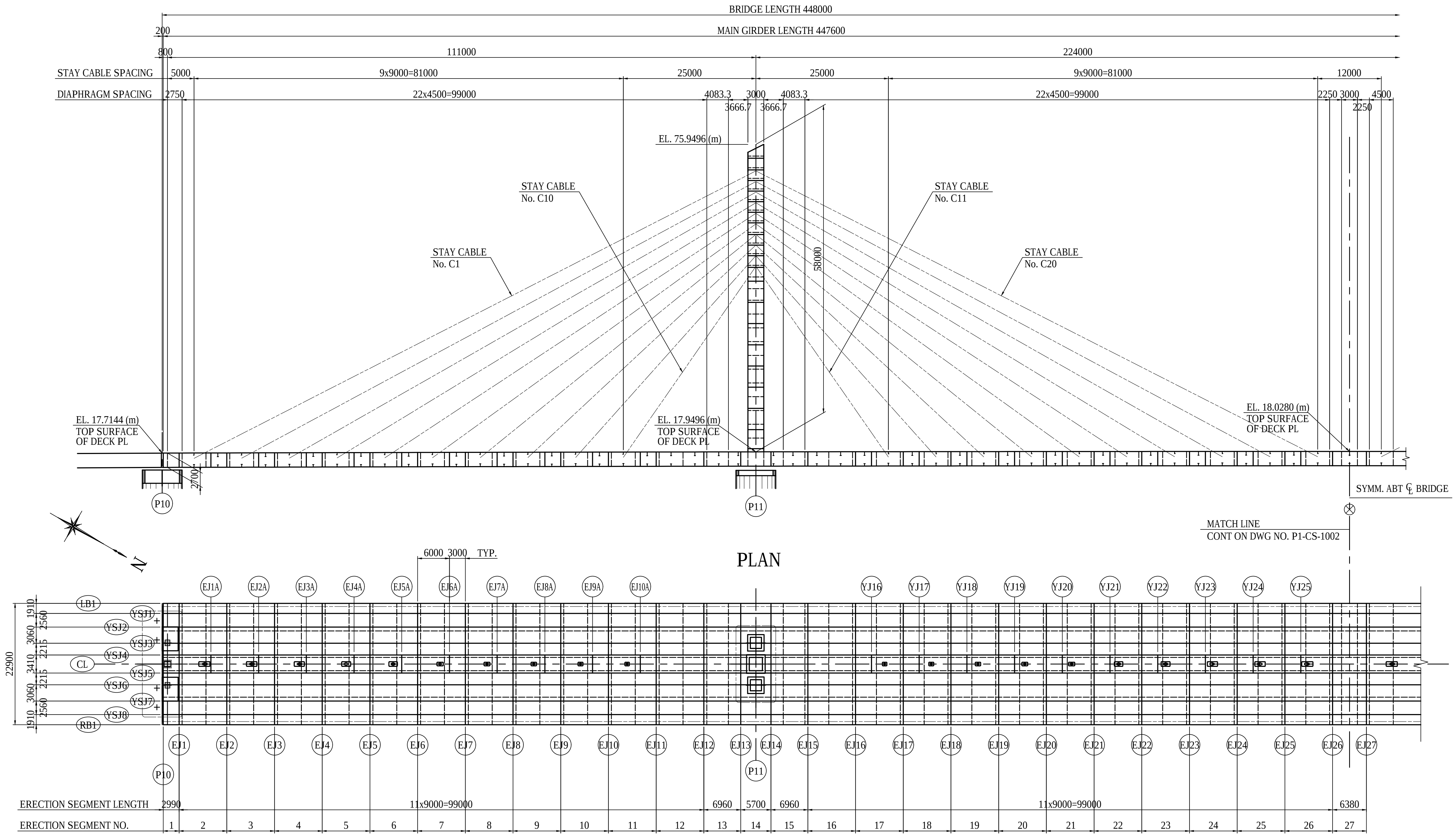
PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.

	NAME	SIGNATURE	DATE
PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017
CHECKED BY	T. HAYAKAWA	早川知邦	28. Nov.2017
APPROVED BY	Y. SANO	佐野 隆一	29. Nov.2017

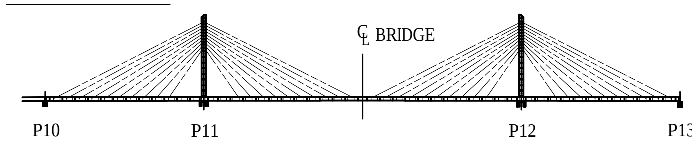
DRAWING TITLE	PACKAGE
GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (3)	1
	DWG No.
	P1-CS-0003

GENERAL VIEW OF MAIN GIRDER (1) S=1:800

ELEVATION

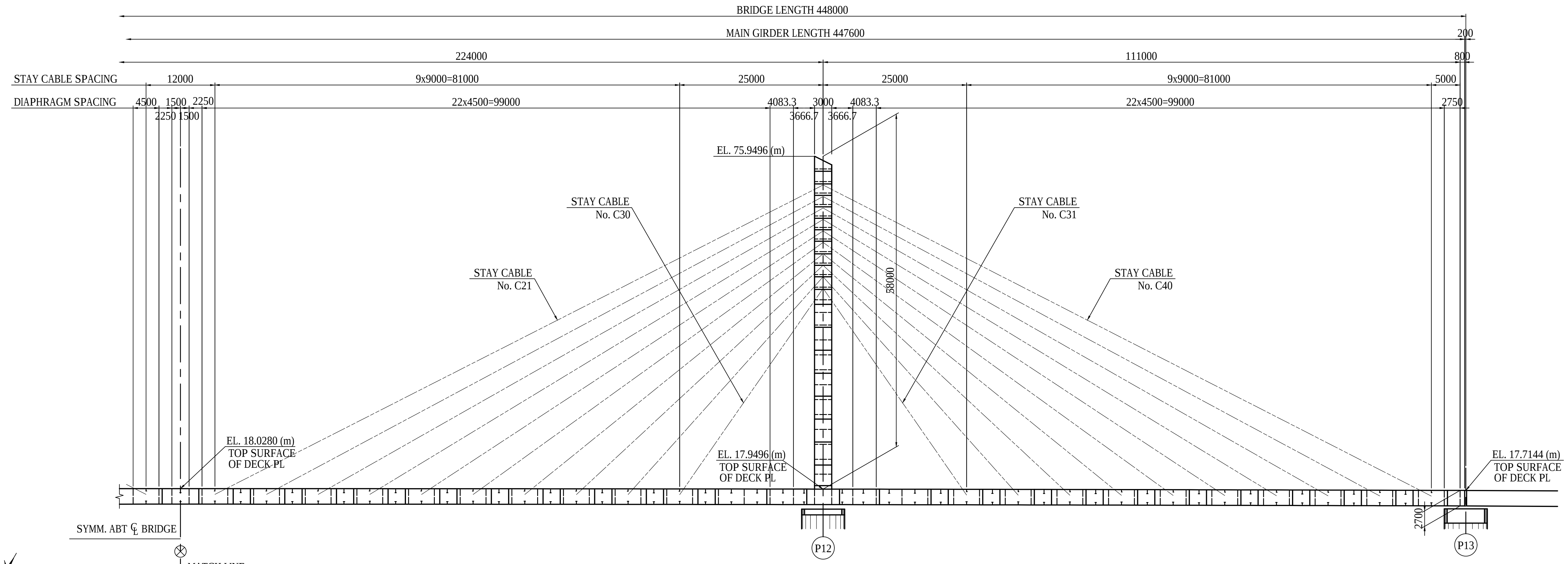


KEY PLAN

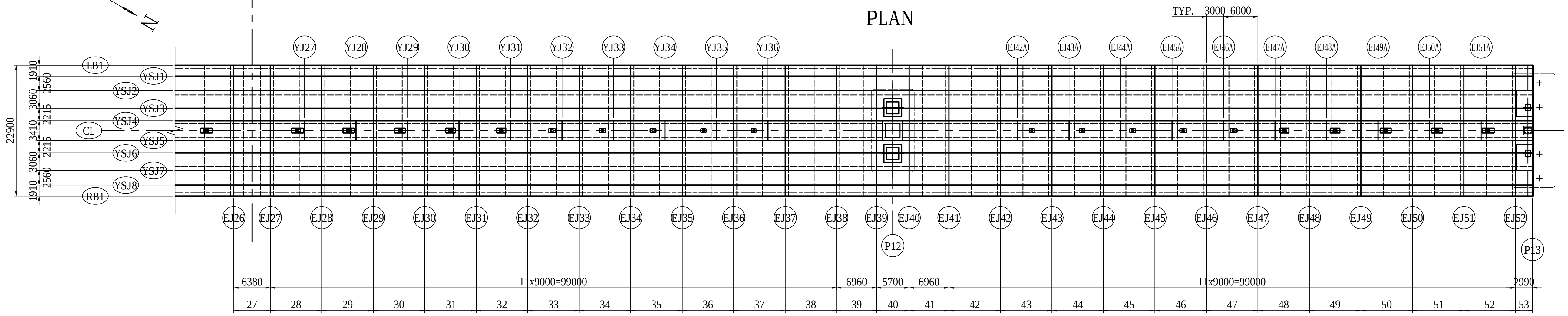


PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA	[signature]	27. Nov.2017	GENERAL VIEW OF MAIN GIRDER (1)	1
				CHECKED BY	T. HAYAKAWA	[signature]	28. Nov.2017		DWG No.
				APPROVED BY	Y. SANO	[signature]	29. Nov.2017		P1-CS-1001

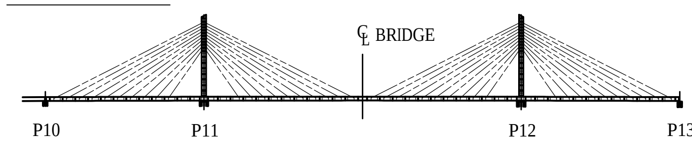
GENERAL VIEW OF MAIN GIRDER (2) S=1:800



PLAN



KEY PLAN



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
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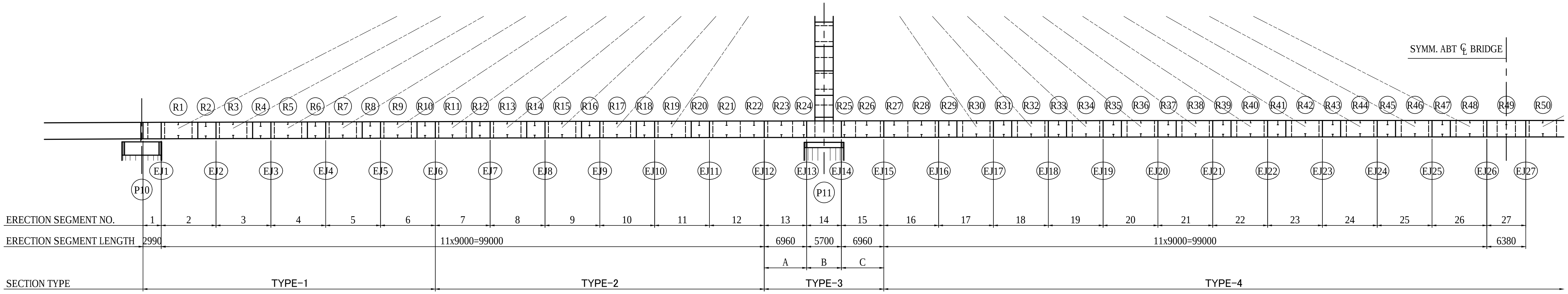
	NAME	SIGNATURE	DATE
PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017
CHECKED BY	T. HAYAKAWA	[signature]	28. Nov.2017
APPROVED BY	Y. SANO	[signature]	29. Nov.2017

DRAWING TITLE	PACKAGE
GENERAL VIEW OF MAIN GIRDER (2)	1
	DWG No.
	P1-CS-1002

GENERAL VIEW OF MAIN GIRDER (8)

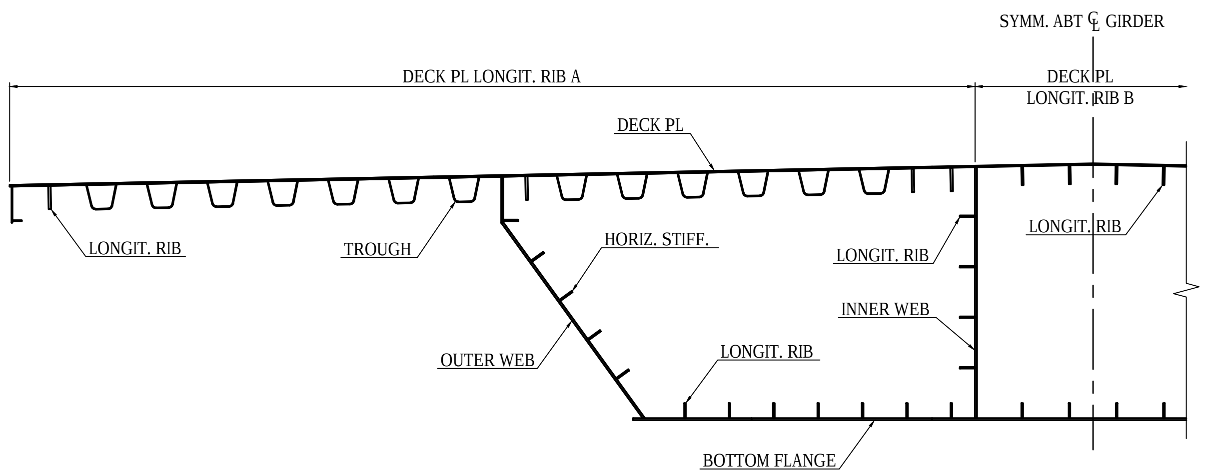
ELEVATION N.T.S.



UNIT： mm

		TYPE-1	TYPE-2	TYPE-3	TYPE-4
DECK PL	THK	16	16	16	16
	STEEL GRADE	SM400	SM400	SM400 / SM490Y / SM400	SM400
	TROUGH	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8
	LONGIT. RIB A	250 x 24	250 x 24	250 x 24	250 x 24
	LONGIT. RIB B	200 x 20	200 x 20	200x20	200 x 20
OUTER WEB	THK	14	14	17	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	HORIZ. STIFF.	160 x 16	160 x 16	200 x 20	160 x 16
INNER WEB	THK	14	14	18 / 35 / 18	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16
BOTTOM FLANGE	THK	14	11	15	11
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16

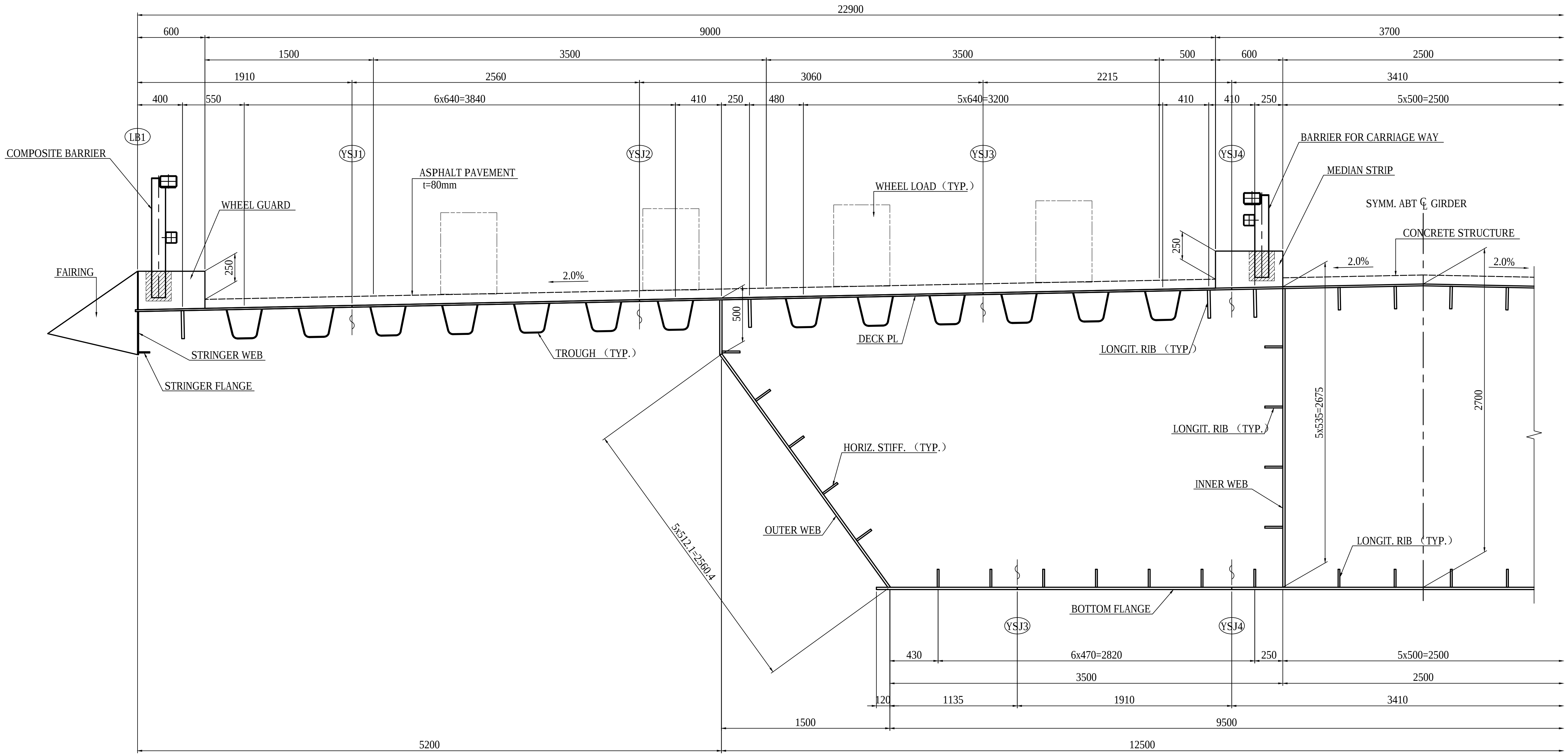
TYPICAL SECTION N.T.S.



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017	GENERAL VIEW OF MAIN GIRDER (8)	1
				CHECKED BY	T. HAYAKAWA	早川知邦	28. Nov.2017		DWG No.
				APPROVED BY	Y. SANO	佐野 雄一	29. Nov.2017		P1-CS-1008

GENERAL VIEW OF MAIN GIRDER (9)   S=1:40

SECTION OUTLINES
TYPICAL SECTION



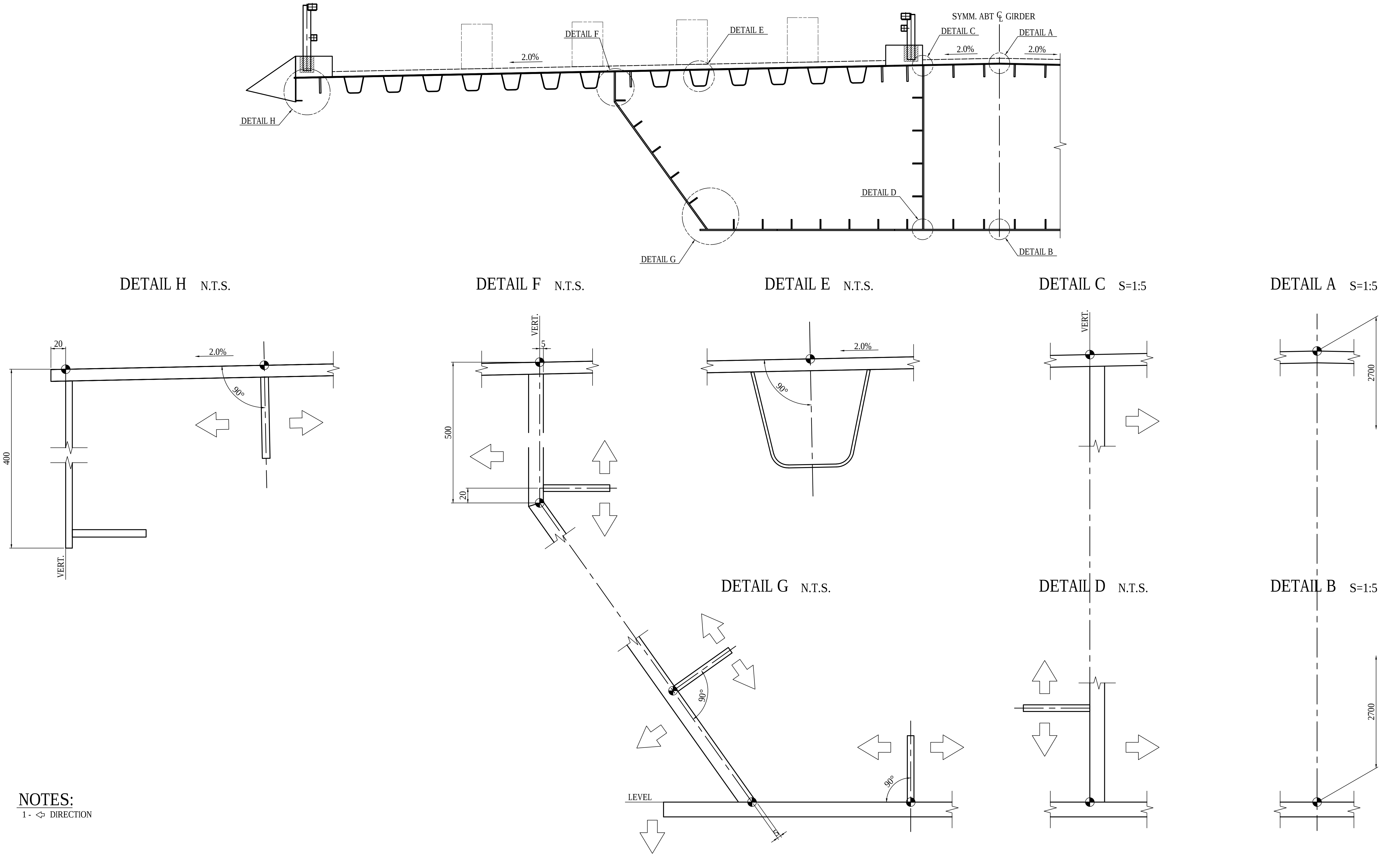
PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.

	NAME	SIGNATURE	DATE
PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017
CHECKED BY	T. HAYAKAWA	早川知邦	28. Nov.2017
APPROVED BY	Y. SANO	佐野裕一	29. Nov.2017

DRAWING TITLE	PACKAGE
GENERAL VIEW OF MAIN GIRDER (9)	1
	DWG No.
	P1-CS-1009

GENERAL VIEW OF MAIN GIRDER (10) S=1:60

SECTION OUTLINES



NOTES:

1 - ⇦ DIRECTION

PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017	GENERAL VIEW OF MAIN GIRDER (10)	1
				CHECKED BY	T. HAYAKAWA	早川知邦	28. Nov.2017		DWG No.
				APPROVED BY	Y. SANO	佐野祐一	29. Nov.2017		P1-CS-1010

Cable Stayed Bridge

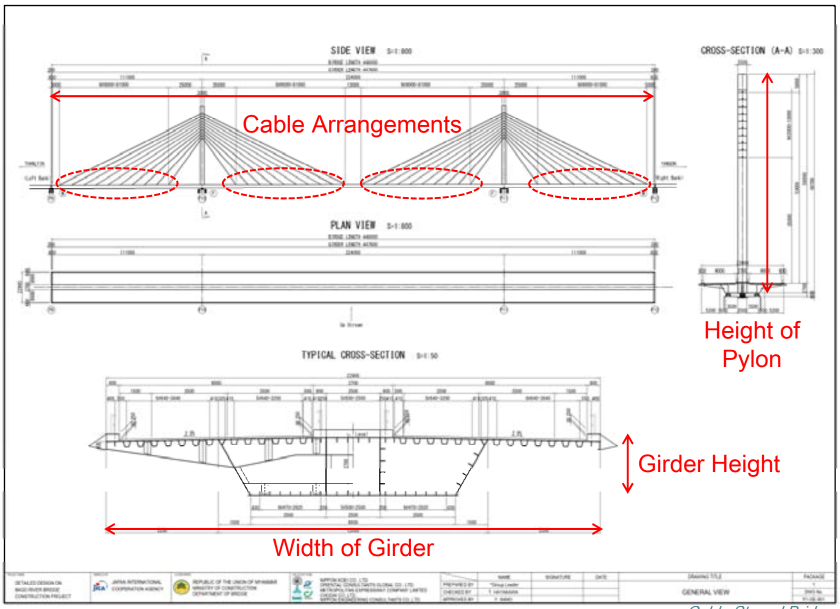
Basic Design-1

Contents for Cable Stayed Bridge

1. B/D Results and Revised Points from F/S
2. Review of F/S Design (Bridge Type)
3. Flow Chart of B/D for Cable Stayed Bridge
4. Selection of Structure Type

JICA Survey Team 3

1. B/D Results and Revised Points from F/S



Cable Stayed Bridge (P9-12)

JICA Survey Team 4

1. B/D Results and Revised Points from F/S

Items		F/S Design	B/D Results
Main Girder			
	Width	24.0m	22.0m
	Height	3.0m	2.7m
Main Tower			
	Height	57.0m (3m+31.5m+9@2m+4.5m)	60.7m (2.7m+35m+9@2m+5m)
	Width	2.4m	2.5m
Cable Arrangements			
	Side Span	12.5m+9@8m+27.5m	5m+9@9m+25m
	Center Span	27.5m+9@8m+12.5m(CL)	25m+9@9m+6m(CL)
Substructure			
	Shape of Pier, Arrangement of Pile, Dimension, etc. were revised.		

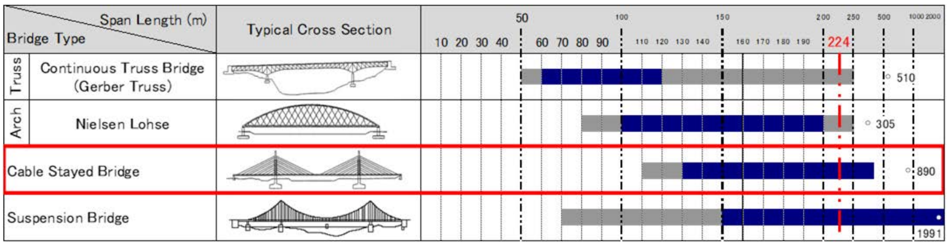
Cable Stayed Bridge (P9-12)

JICA Survey Team 5

2. Review of F/S Design (Bridge Type)

In case of Main Span length L=224m, following type of bridges can be applied. However, because of the reasons below, only a Cable Stayed Bridge can be applied in this Project.

- Truss Bridge : Gerber Truss should be used and not good at maintenance and cost.
- Arch Bridge : There are no applicable construction methods in this Project.
- Suspension Bridge : Anchorage can't be constructed in the river.



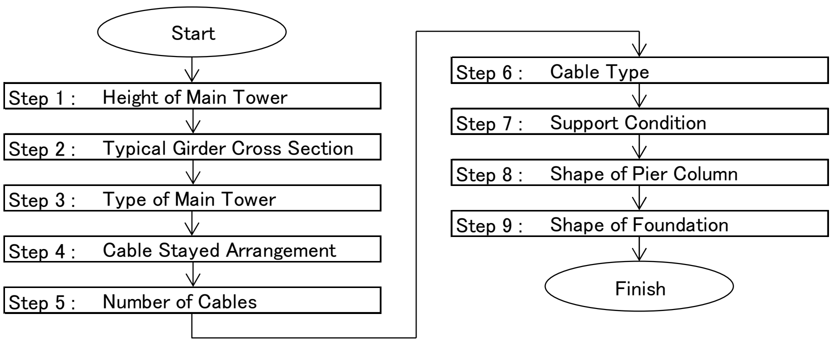
Cable Stayed Bridge (P9-12)

JICA Survey Team 6

3. Flow Chart of B/D for Cable Stayed Bridge

In the B/D for Cable Stayed Bridge, 9 Important Items were studied as shown in the flow chart.

At each step, basically 3-alternatives were compared and the best type was selected.



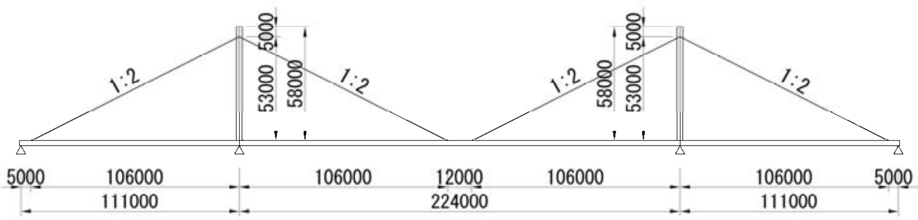
Cable Stayed Bridge (P9-12)

4. Selection of Structure Type

Step1: Height of Main Tower

Generally, most economical gradient of top cable at cable stayed bridge is 1:2.

Considering side span length, the gradient of top cable and working space at top of tower, height of main tower was decided as 58m in this Project.

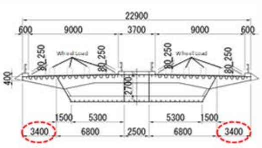
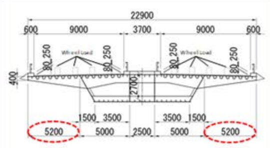
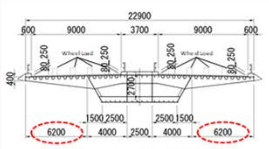


Cable Stayed Bridge (P9-12)

Step2: Typical Girder Cross Section

Comparison of typical girder cross section is shown as follows.

“Conventional Box Cross Section” was selected as the best one.

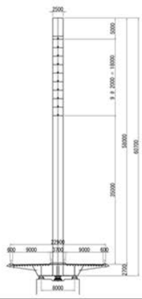
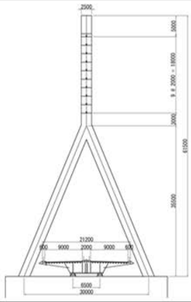
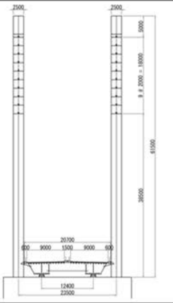
Type	① : Wide Box Cross Section	② : Conventional Box Cross Section	③ : Narrow Box Cross Section
Figure			
Characteristics	- Web is not located under the wheel load, therefore this type is good at fatigure resistance. - Due to the wide box cross section, the number of parts of the girder will be increased, and increasing assembly time at Site.	- Web is not located under the wheel load, therefore this type is good at fatigue resistance. - Steel weight is lower than other types, therefore this cross section is the most economical.	- Web is located under the wheel load, therefore this type is not good at fatigue resistance. - Overhang length is large, therefore this type is not good at deflection and fatigue resistance.
Cost Ratio	1.01	**1.00**	1.01
Evaluation		o	

(In the F/S Stage, “Conventional Box Cross Section” was selected same with the B/D. [at the F/S Stage, overhang length is 5400mm])

Cable Stayed Bridge (P9-12)

Step3: Type of Main Tower

Comparison of type of main tower is shown as follows.
"Single Tower" was selected as the best one.

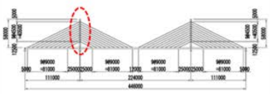
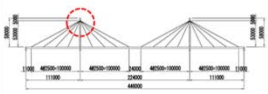
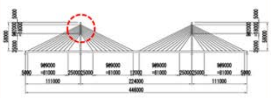
Type	① : Single Tower	② : A-Shape Tower	③ : Twin Tower
Figure			
Characteristics	- This type has 1 straight pylon, and it's a simple structure rather than others.	- Column of main tower is located at both sides of girder, therefore pier width will be wider.	- Column of main tower is located at both sides, therefore compared to type-1, pier width will be wider.
Cost Ratio*	1.00	1.23	1.17
Evaluation	o		

* This total cost ratio includes cost of superstructure and substructure.
(In the F/S Stage, "Single Tower" was selected same with the B/D)

Cable Stayed Bridge (P9-12)

Step4: Cable Stayed Arrangement

Comparison of cable stayed arrangement is shown as follows.
"Semi Fan Arrangement" was selected as the best one.

Type	① : Harp Arrangement	② : Fan Arrangement	③ : Semi Fan arrangement
Figure	 Each cable is stayed at whole part	 All cables are stayed at top	 Cables are distributed at upper part
Characteristics	- From structural point of view, the lower cables are not so efficient due to the gradient . - From aesthetic point of view, it has a good appearance and it's more attractive than other arrangement types.	- All cables are attached to single point at the top of the pylon, making difficult to attach the cables to one point, therefore this type is not applied in long span cable stayed bridges.	- From structural point of view, gradient of the lower cables are bigger than Harp Arrangement, therefore structural efficiency is higher. - From aesthetic point of view, cables are arranged by a single plane, giving also good appearance.
Cost Ratio	1.05	1.01	1.00
Evaluation			o

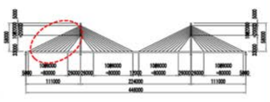
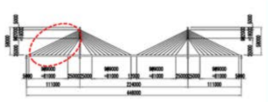
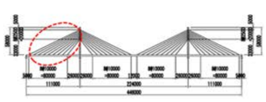
(In the F/S Stage, "Semi Fan Arrangement" was selected same with the B/D)

Cable Stayed Bridge (P9-12)

JICA Survey Team 11

Step5: Number of Cables

Comparison of number of cables is shown as follows.
"10 Cables" was selected as the best one.

Type	① : 11 Cables	② : 10 Cables	③ : 9 Cables
Figure	 11 cables at one-side, total 44 cables	 10 cables at one-side, total 40 cables	 9 cables at one-side, total 36 cables
Characteristics	- For each cable installed, the stiffening girder and the pylon need to be strengthen locally in order to be able to receive the stayed forces. Therefore, increasing the cables also increases the parts that require local strengthening.	- This cable arrangement is more economical, due to the lower weight of the total cables compared to the 9 cables arrangement.	- Due to the increase of space between cables, the length and weight of the blocks increases, making neccesary a bigger crane for the erection work.
Cost Ratio	1.01	1.00	1.02
Evaluation		o	

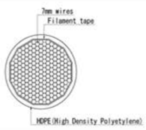
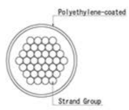
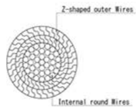
(In the F/S Stage, "10 Cables" was selected same with the B/D)

Cable Stayed Bridge (P9-12)

JICA Survey Team 12

Step6: Type of Cables

Comparison of type of cables is shown as follows.
"FUT-H strand cables" was selected as the best one.

Type	① : NPWS(New Parallel Wire Strand)	② : FUT-H strand cables	③ : Locked Coil Rope
Figure			
Young's modulus	1.95×10^5 N/mm^2	1.90×10^5 N/mm^2	1.55×10^5 N/mm^2
Characteristics	- NPWS cables are prefabricated at factory, reducing the erection time of the cables in site. - In order to install this type of cable it's necessary heavy machinery, such as big cranes and jacks.	- FUT-H cables are formed in site by tensioning each stranded wire one by one, slightly increasing the time of erection. - During installation it's necessary a small crane, jacks and other small machines, avoiding the use of big cranes and vast loads on the girder during erection.	- LCR cables are prefabricated at factory, but in order to apply this type of cable in this design there where neccesary 3 cables per section, increasing the time of erection and fabrication.
Cost Ratio	1.25	1.00	1.05
Evaluation		o	

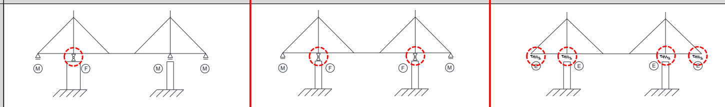
(In the F/S Stage, Type-1 "NPWS" was selected)

Cable Stayed Bridge (P9-12)

JICA Survey Team 13

Step7: Support Condition

Comparison of support condition is shown as follows.
"M-F-F-M" support condition was selected as the best one.

Type	① : M-F-M-M	② : M-F-F-M	③ : E-E-E-E
Figure	 Pin bearing is used at 1-main pier	Pin bearing is used at 2-main piers	Rubber bearing is used at all of pier
Characteristics	- Horizontal displacement is resisted by pin bearing at 1 main tower, therefore horizontal displacement of main girder due to temperature change and earthquake becomes slightly large.	- Rigidity of overall structure is high, and horizontal movement is fixed by pin bearing to the main tower.	- Horizontal displacement is resisted by rubber bearing at the 2 main towers and the 2 side piers at both ends, therefore horizontal displacement of main girder due to earthquake becomes large.
	- In order to fix horizontal movement of girder, temporary pin bearing supports should be installed at M-supported pier at main tower during erection works.	- Horizontal displacement under earthquakes is small, therefore gap at girder ends and expansion joints become compact.	- Horizontal force caused by earthquake is dispersed throughout the piers, therefore this type is efficient for seismic design.
Displacement	Temperature : -60mm~+40mm	Temperature : -40mm~+30mm	Temperature : -40mm~+30mm
	Earthquake : 150mm	Earthquake : 70mm	Earthquake : 650mm
Cost Ratio*	1.02	1.00	1.02
Evaluation		o	

* This total cost ratio includes cost of superstructure and substructure.

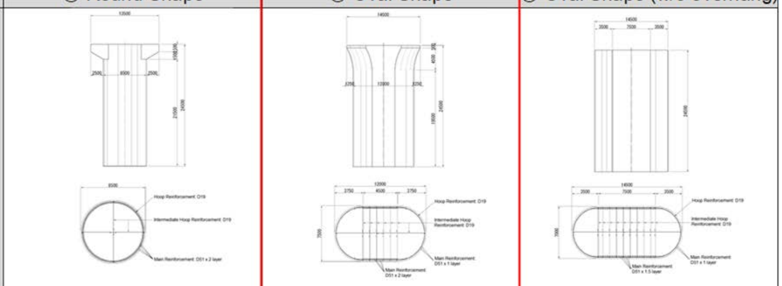
(In the F/S Stage, Type-3 "E-E-E-E" support condition was selected)

Cable Stayed Bridge (P9-12)

JICA Survey Team 14

Step8: Shape of Pier Column (P10, P11)

Comparison of shape of pier column is shown as follows.
"Oval Shape" was selected as the best one.

	① Round Shape	② Oval Shape	③ Oval Shape (w/o overhang)
Cross Section			
Characteristics	- Width of column at transverse direction is wider than other types, therefore impediment ratio of river flow become big.	- From aesthetic point of view, since this type has same shape with other spans, it has a good appearance.	- Because of the big cross section, concrete volume become large.
Cost Ratio	0.87	1.00	1.01
Evaluation	×(Pier column at river should be oval shape)	o	

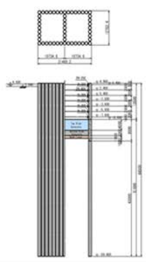
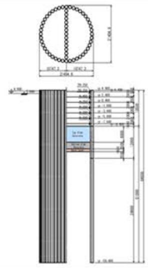
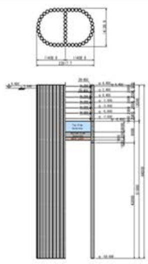
(In the F/S Stage, Type-3 "Oval Shape (w/o over hang)" was selected)

Cable Stayed Bridge (P9-12)

JICA Survey Team 15

Step9: Shape of Foundation (P10, P11)

Comparison of shape of foundation is shown as follows.
"Oval Shape" was selected as the best one.

	① Rectangle Shape	② Round Shape	③ Oval Shape
Cross Section			
Characteristics	- From structural point of view, this rectangle shape is unfavorable to water flow and many support should be installed during construction work.	- Pier shape was decided as "Oval Shape", therefore this type has too much needless space.	- Pier shape was decided as "Oval Shape" and this foundation has same shape, therefore this type is the most reasonable.
Cost Ratio	1.06	1.12	1.00
Evaluation			○

(In the F/S Stage, "Oval Shape" was selected same with the B/D)

Cable Stayed Bridge (P9-12)

JICA Survey Team 16

Cable Stayed Bridge

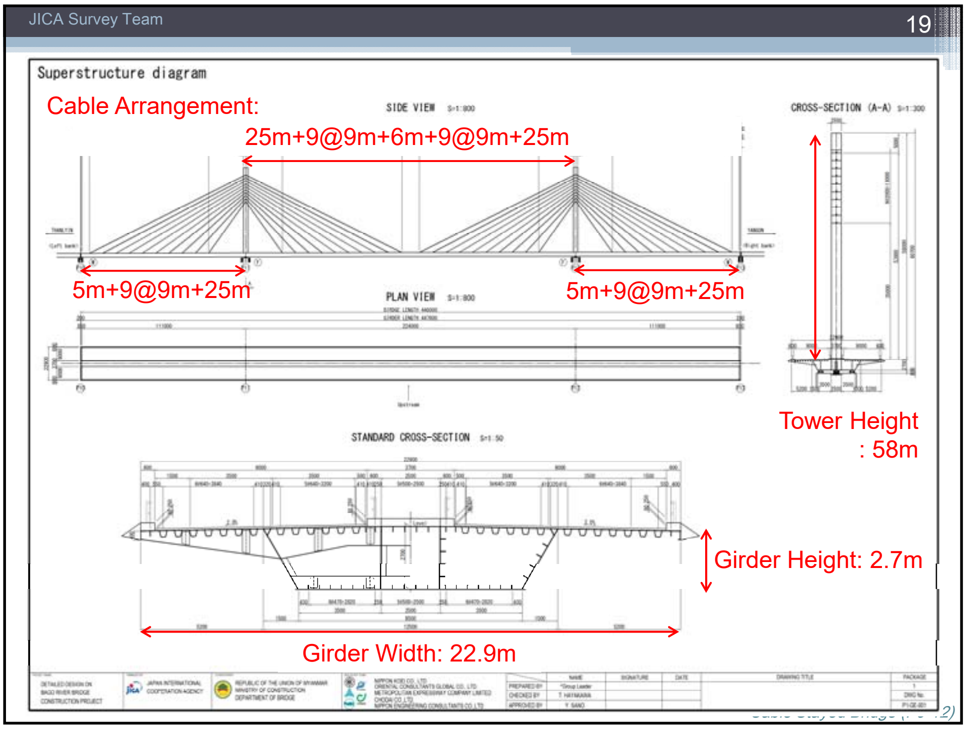
Basic Design-2

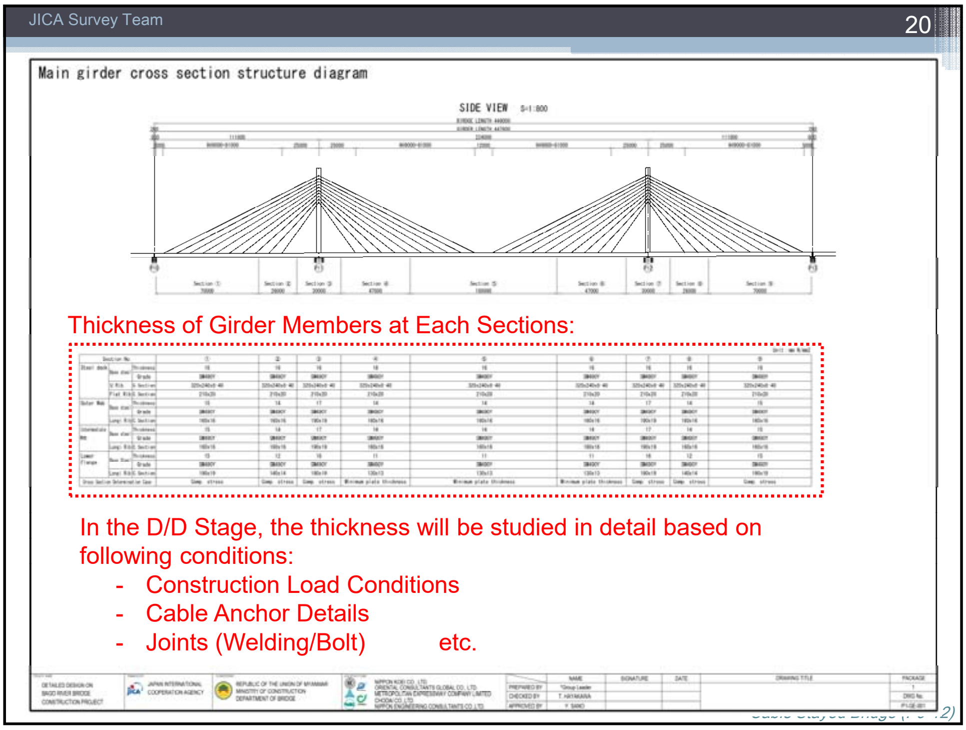
Cable Stayed Bridge (P9-12)

Contents for Cable Stayed Bridge

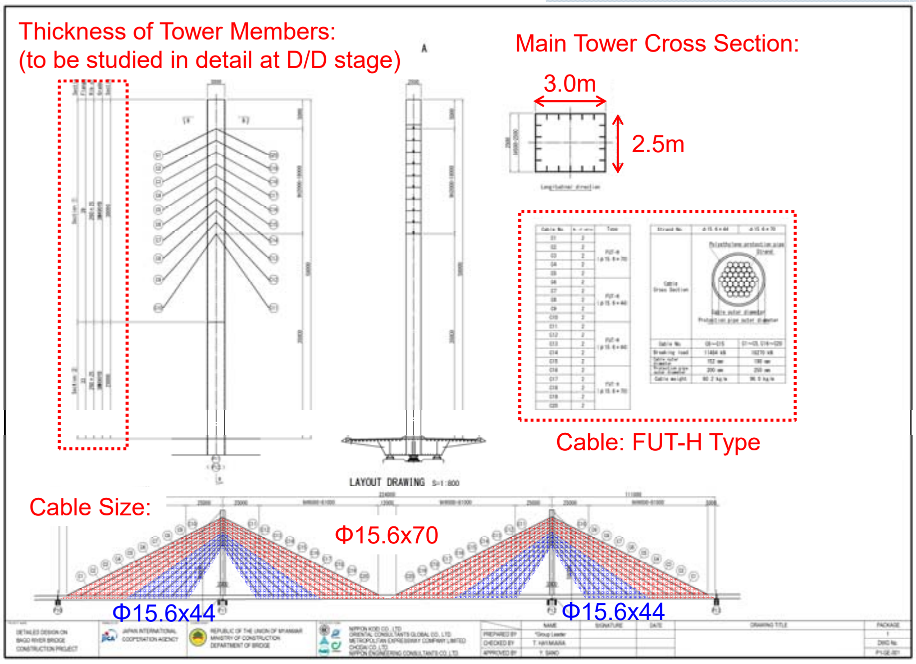
1. Summary of B/D Results
(1) Superstructure

Items		F/S Results	B/D Results
Main Girder			
	Width	24.0m	22.9m
	Height	3.0m	2.7m
Main Tower			
	Height	54.0m	58.0m
	Cross Section	2.4m x 3.0m	2.5m x 3.0m
Cable			
	Type	New PWS	FUT-H Type
	Cross Section	---	Φ15.6 x 44, 70
	Side Span	12.5m+9@8m+27.5m	5m+9@9m+25m
	Center Span	27.5m+9@8m+12.5m(CL)	25m+9@9m+6m(CL)


JICA Survey Team
19
Superstructure diagram
Cable Arrangement:
25m+9@9m+6m+9@9m+25m
5m+9@9m+25m
5m+9@9m+25m
SIDE VIEW
PLAN VIEW
CROSS-SECTION (A-A)
STANDARD CROSS-SECTION
Tower Height
: 58m
Girder Height: 2.7m
Girder Width: 22.9m


JICA Survey Team
20
Main girder cross section structure diagram
SIDE VIEW
Thickness of Girder Members at Each Sections:
In the D/D Stage, the thickness will be studied in detail based on following conditions:
- Construction Load Conditions
- Cable Anchor Details
- Joints (Welding/Bolt) etc.

JICA Survey Team 21



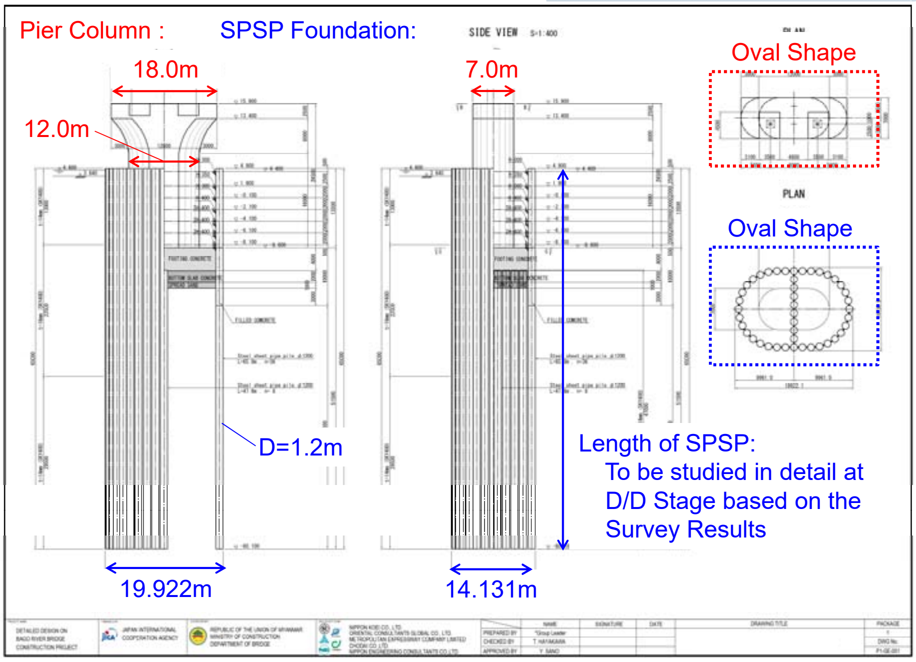
JICA Survey Team 22

1. Summary of BD Results
(2) Substructure at Side Pier (P10, P13)

Items	F/S Results	B/D Results
Side Pier (P10, P13)		
Pier Column		
Shape	Oval Shape	Oval Shape
Cross Section Bottom Pier Head	 4.5m x 14.0m 4.5m x 20.0m	 7.0m x 12.0m 7.0m x 18.0m
SPSP Foundation		
Pile Diameter	1.0m	1.2m
Shape	Oval Shape	Oval Shape
Cross Section	9.768 x 19.750 (m)	14.131 x 19.922 (m)

Cable Stayed Bridge (P9-12)

JICA Survey Team 23



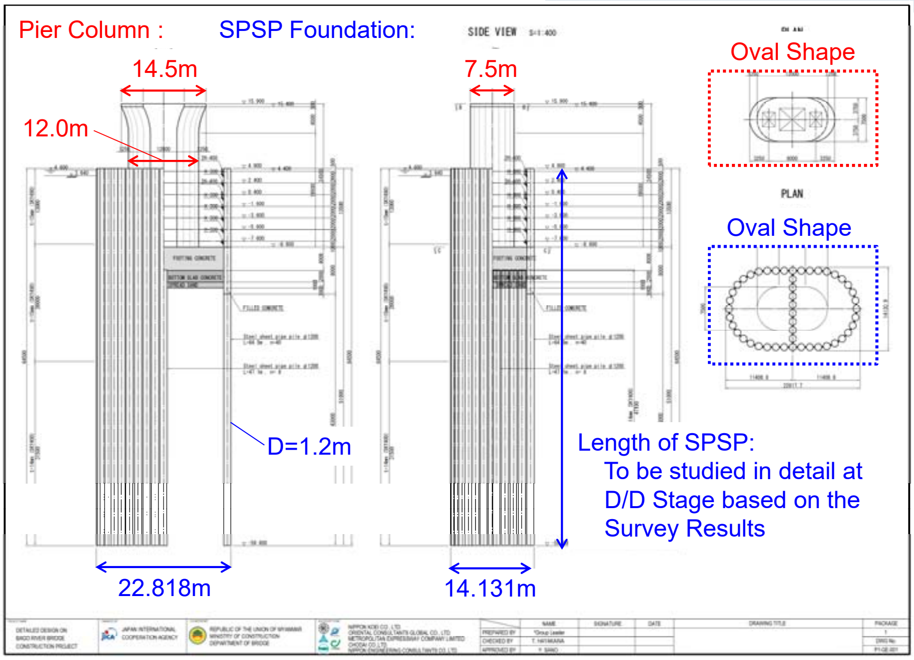
JICA Survey Team 24

1. Summary of BD Results
(3) Substructure at Intermediate Pier (P11, P12)

Items	F/S Results	B/D Results
Intermediate Pier (P11, P12)		
Pier Column		
Shape	Oval Shape (w/o overhang)	Oval Shape
Cross Section Bottom Pier Head	 8.0m x 11.0m 8.0m x 11.0m	 7.5m x 12.0m 7.5m x 14.5m
SPSP Foundation		
Pile Diameter	1.0m	1.2m
Shape	Oval Shape	Oval Shape
Cross Section	13.730 x 16.226 (m)	14.131 x 22.818 (m)

Cable Stayed Bridge (P9-12)

JICA Survey Team 25



JICA Survey Team 26

2. Items to be Studied at D/D Stage

Items	Descriptions
Election Calculation	Verify the safety and deformation state during election works.
Load-bearing Capacity	Consider the non-linear behavior of section force at cable stayed bridge, analyze by [1.7 x design load] and compare with yield stress.
Cable Anchor	3D-FEM Analysis will be conducted in order to check local stress.
Seismic Design	Apply seismic waveforms and analyze time-history response.
Soil Parameter	Update soil parameter based on the Survey results.
River Bed Height	Update river bed height and footing level based on the Survey results.

Cable Stayed Bridge (P9-12)

JICA Survey Team 1

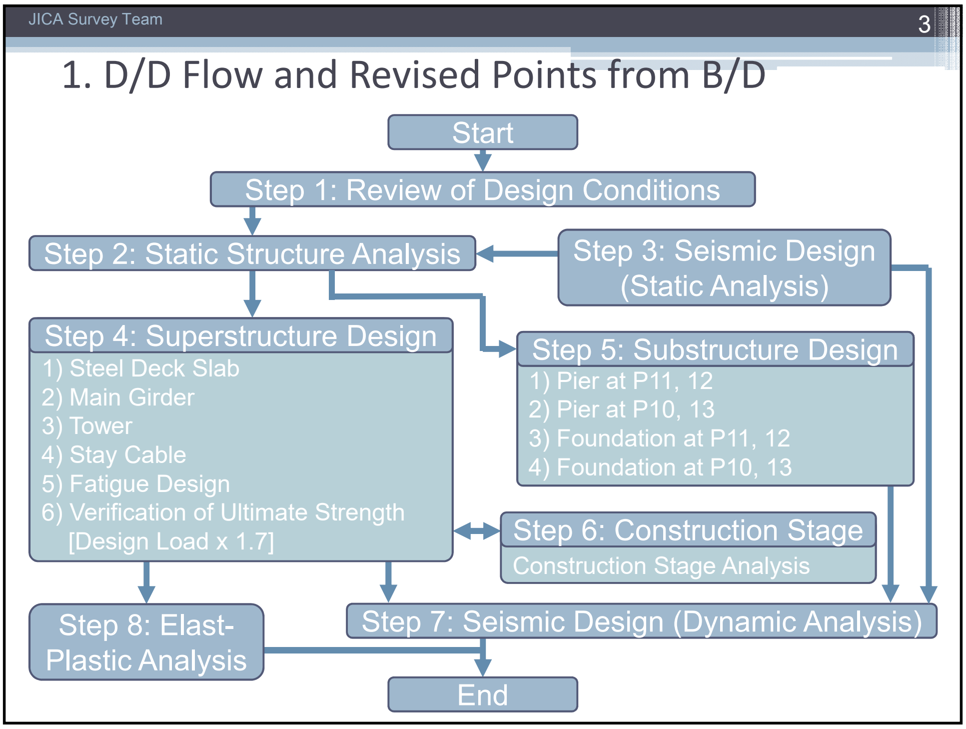
Cable Stayed Bridge

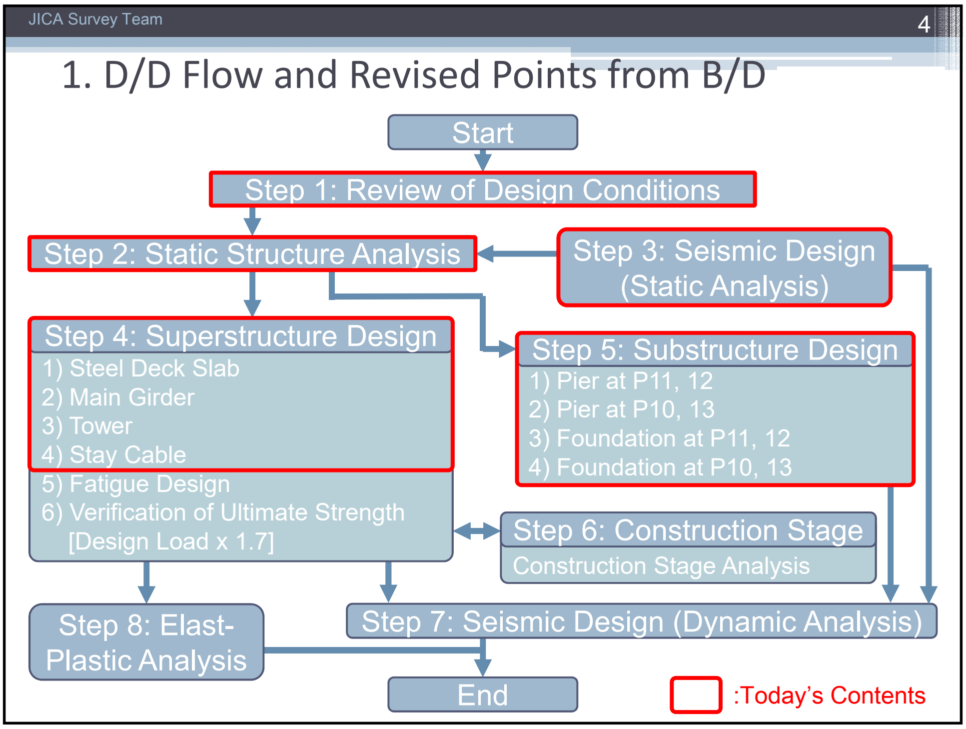
Detailed Design-1

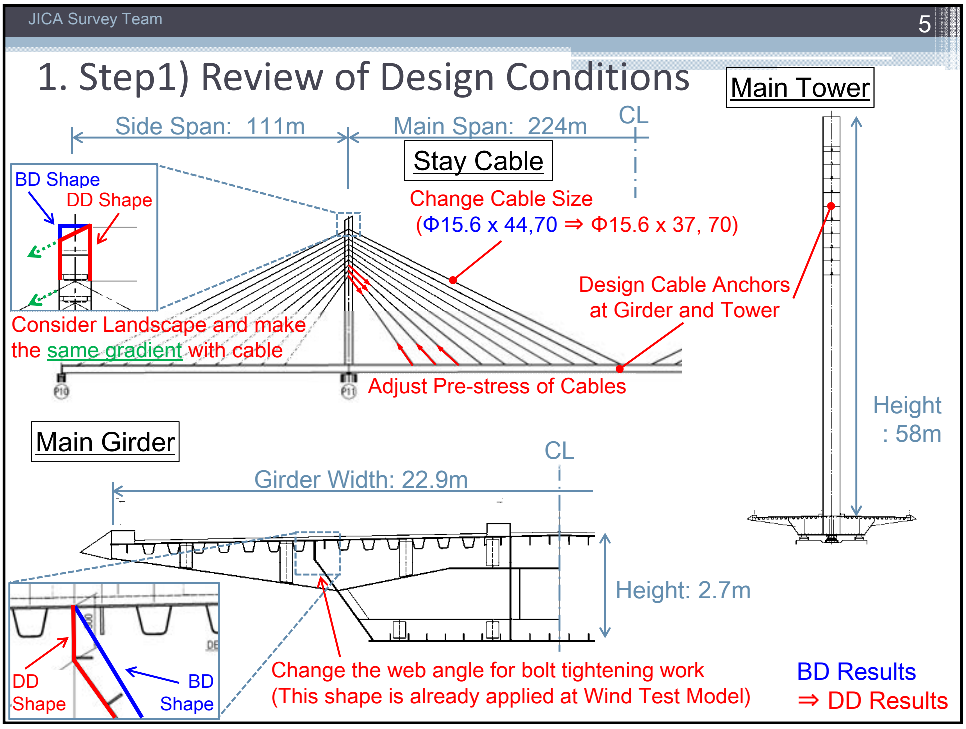
JICA Survey Team 2

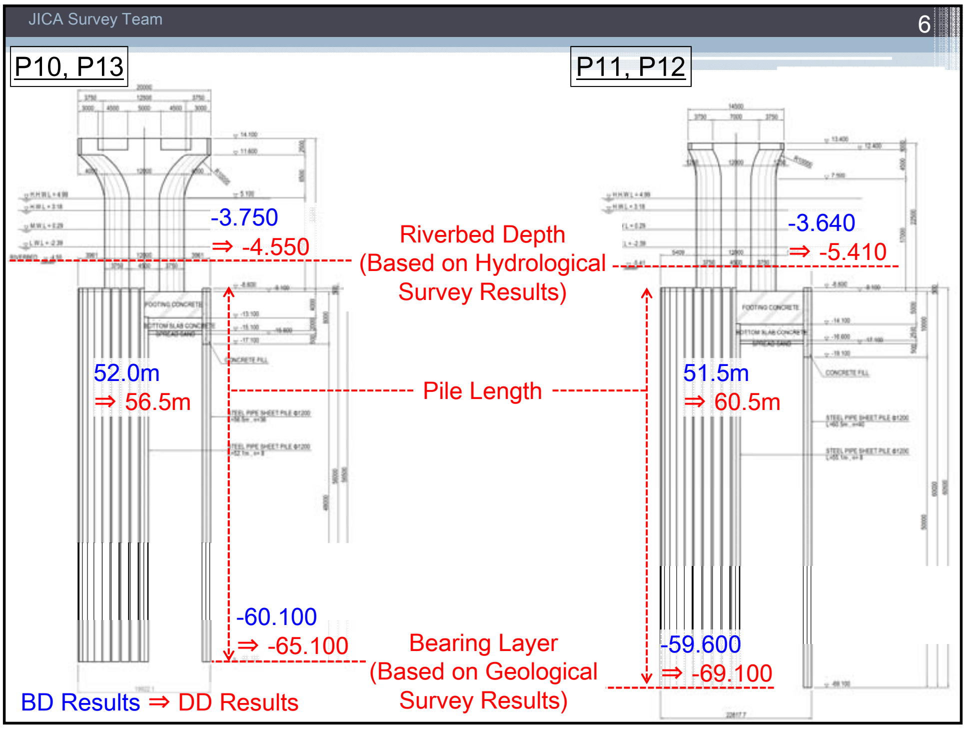
Contents for Cable Stayed Bridge

1. D/D Flow and Revised Points from B/D
2. D/D Results of Superstructure
3. D/D Results of Substructure
4. Items to be Reported in the Next Meeting


JICA Survey Team
3
1. D/D Flow and Revised Points from B/D
Start
Step 1: Review of Design Conditions
Step 2: Static Structure Analysis
Step 3: Seismic Design (Static Analysis)
Step 4: Superstructure Design
1) Steel Deck Slab
2) Main Girder
3) Tower
4) Stay Cable
5) Fatigue Design
6) Verification of Ultimate Strength [Design Load x 1.7]
Step 5: Substructure Design
1) Pier at P11, 12
2) Pier at P10, 13
3) Foundation at P11, 12
4) Foundation at P10, 13
Step 6: Construction Stage
Construction Stage Analysis
Step 8: Elast-Plastic Analysis
Step 7: Seismic Design (Dynamic Analysis)
End


JICA Survey Team
4
1. D/D Flow and Revised Points from B/D
Start
Step 1: Review of Design Conditions
Step 2: Static Structure Analysis
Step 3: Seismic Design (Static Analysis)
Step 4: Superstructure Design
1) Steel Deck Slab
2) Main Girder
3) Tower
4) Stay Cable
5) Fatigue Design
6) Verification of Ultimate Strength [Design Load x 1.7]
Step 5: Substructure Design
1) Pier at P11, 12
2) Pier at P10, 13
3) Foundation at P11, 12
4) Foundation at P10, 13
Step 6: Construction Stage
Construction Stage Analysis
Step 8: Elast-Plastic Analysis
Step 7: Seismic Design (Dynamic Analysis)
End
:Today's Contents


JICA Survey Team
5
1. Step1) Review of Design Conditions
Main Tower
Side Span: 111m
Main Span: 224m
CL
Stay Cable
BD Shape
DD Shape
Change Cable Size
(Φ15.6 x 44,70 ⇒ Φ15.6 x 37, 70)
Design Cable Anchors
at Girder and Tower
Consider Landscape and make
the same gradient with cable
Adjust Pre-stress of Cables
Height
: 58m
Main Girder
CL
Girder Width: 22.9m
Height: 2.7m
DD
Shape
BD
Shape
Change the web angle for bolt tightening work
(This shape is already applied at Wind Test Model)
BD Results
⇒ DD Results

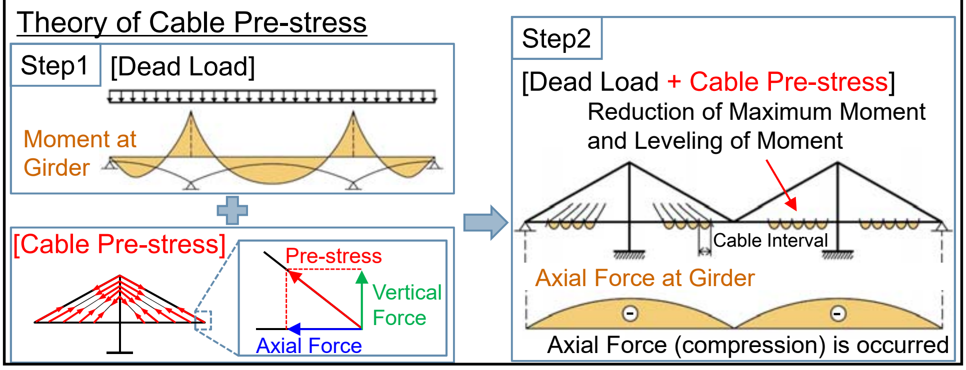

JICA Survey Team
6
P10, P13
P11, P12
-3.750
⇒ -4.550
Riverbed Depth
(Based on Hydrological
Survey Results)
-3.640
⇒ -5.410
52.0m
⇒ 56.5m
Pile Length
51.5m
⇒ 60.5m
-60.100
⇒ -65.100
Bearing Layer
(Based on Geological
Survey Results)
-59.600
⇒ -69.100
BD Results ⇒ DD Results

JICA Survey Team 7

2. Step2,3,4) D/D Results of Superstructure

1) Cable Pre-stress

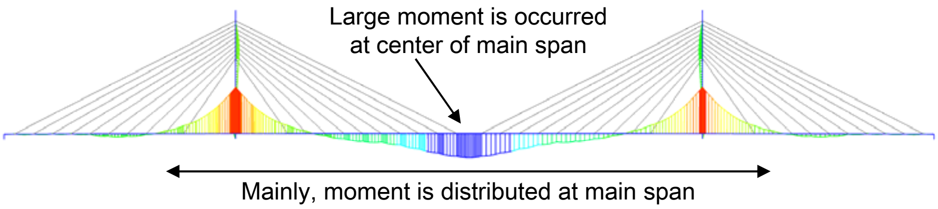
- "How to decide Cable Pre-stress" is the most essential issue for Cable Stayed Bridge Design.
- In this Project, Cable Pre-stress was decided based on following conditions at construction completed state;
  1) Leveling of Moment at Girder
  2) Moment at Tower Base ≒ 0



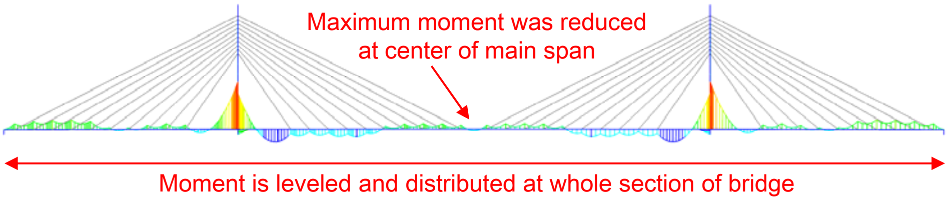
JICA Survey Team 8

2) Moment Diagram at Dead Load and Pre-stress conditions

Step1: [Dead Load]



Step2: [Dead Load + Cable Pre-stress]

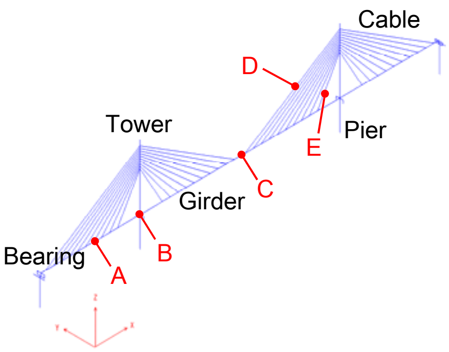


JICA Survey Team 9

3) Static Structure Analysis

- Modeling the Bridge by Beam/Truss (Cable) Elements.
- Apply all Design Loads (Dead Load, Live Load, Wind, Seismic and etc.) to the Model and start static analysis.
- Output "Stress Resultants" of each elements.

Analysis Model



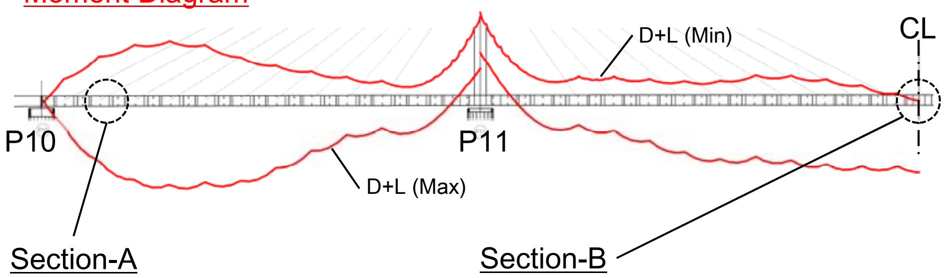
Stress Resultants

Item	Point	Load Case	Axial [kN]	Shear_Y [kN]	Shear_Z [kN]	Torsion [kN· m]	Moment_Y [kN· m]	Moment_Z [kN· m]
Main Girder	A	D+L (max)	-18330.7	464.4	17.0	7934.2	42914.0	741.5
		D+ELG (max)	-10341.7	-599.8	0.6	-5.9	4954.8	-33.8
	B	D+L (max)	-22667.2	-3421.8	15.0	23709.3	-24637.1	1685.3
		D+ELG (max)	-13246.1	2570.3	0.6	-5.8	715.0	-74.2
	C	D+L (max)	4088.7	1346.4	13.8	11009.8	32766.5	435.3
		D+ELG (max)	1574.5	82.3	-0.4	-0.1	1567.1	-26.5
Cable	D	D+PS	4511	---	---	---	---	---
		D+PS+L (max)	5794	---	---	---	---	---
	E	D+PS	2721	---	---	---	---	---
		D+PS+L (max)	3527	---	---	---	---	---

JICA Survey Team 10

3) Design Results of Main Girder and Moment Diagram

Moment Diagram



Section-A

Item	Material	Plate Thickness	Stress: Load Case	Stress: σ or τ	Stress: Allowable Value
Deck	SM400	16mm	D+L+T	52.3	140
Web	SM490Y	14mm	D+L	25.8	120
Bottom Flange	SM490Y	14mm	D+L+T	-94.5	146

Section-B

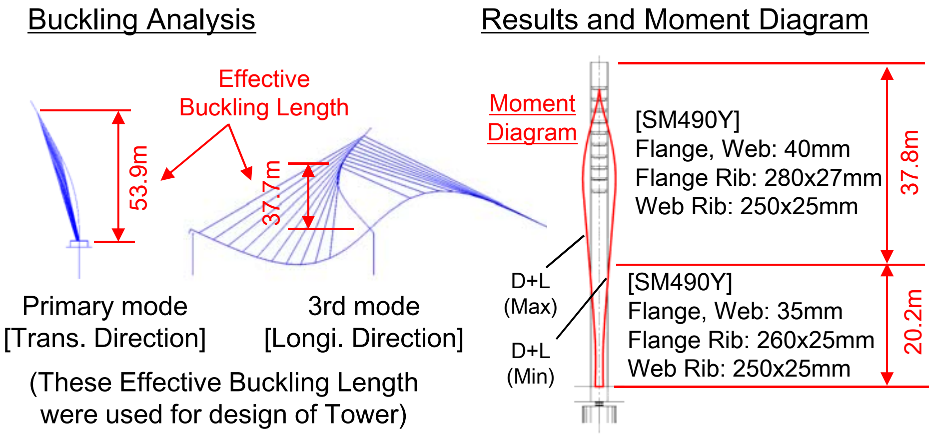
Item	Material	Plate Thickness	Stress: Load Case	Stress: σ or τ	Stress: Allowable Value
Deck	SM400	16mm	D+ETR	45.6	140
Web	SM490Y	14mm	D+L	31.4	120
Bottom Flange	SM490Y	11mm	D+ETR	-86.7	102

Deck PL is determined based on fatigue and Web PL is by Cable anchor.

JICA Survey Team 11

4) Design Results of Main Tower and Buckling

- Bending moment and Axial force affect Tower Design and Tower has slender shape, therefore “Global Buckling” should be prevented.
- Check buckling mode shape by buckling analysis.



JICA Survey Team 12

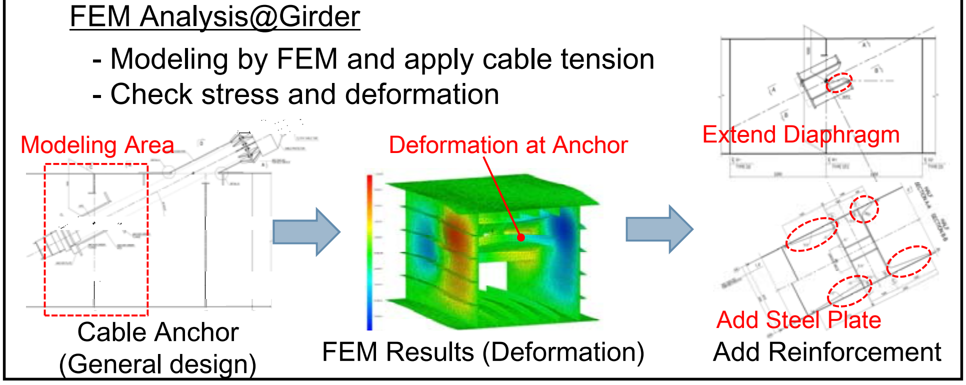
5) FEM Analysis for Cable Anchor

- Cable is supported by Cable Anchor at girder and tower.
- Cable Anchor receive Cable Tension and transmit the tension to girder and tower.
- Cable Anchor Area will become complicated.

1) Verify stress concentration by FEM around Cable Anchor
2) Apply a necessary reinforcement.

FEM Analysis@Girder

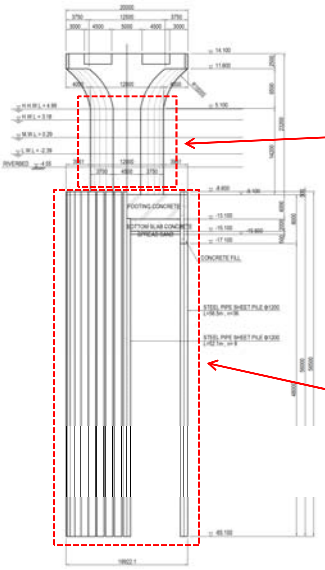
- Modeling by FEM and apply cable tension
- Check stress and deformation



JICA Survey Team 13

3. Step5) D/D Results of Substructure

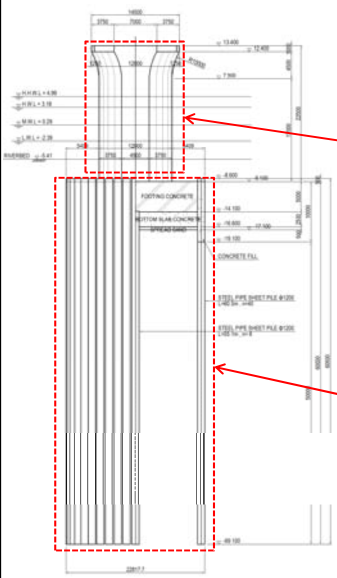
1) Design Results at P10, 13



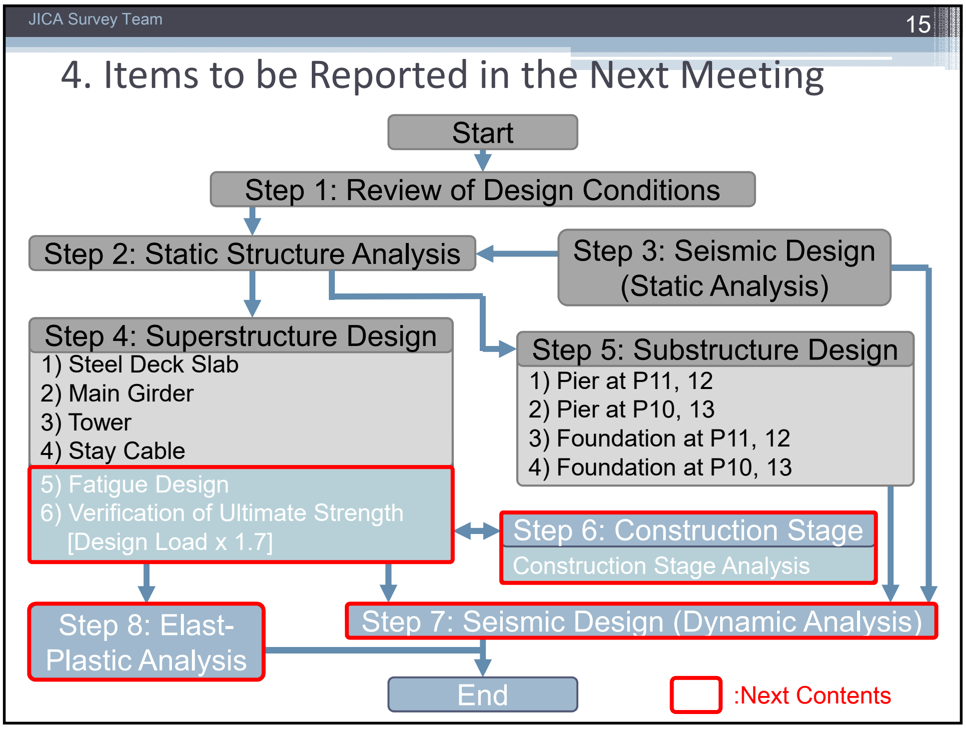
						Longitudinal				Transverse			
Column	Section	Cross Section			m	Oval	;	12.000	×	7.500			
		Re-bar	Main Re-bar	1st				D32	ctc	125			
				2nd				D32	ctc	125			
			Hoop	---				D22	ctc	150			
	Calculation	Earthquake (EQ)	σc		N/mm2	7.67	≦	15.00	○	5.67	≦	15.00	○
			σs		N/mm2	248.0	≦	300.0	○	134.5	≦	300.0	○
			τm		N/mm2	0.276	>	0.152	-	0.259	>	0.136	-
			Aw_req		mm2	789.6	≦	3096.8	○	494.2	≦	2322.6	○
SPSP	Section	D(mm)×L(m)×nos(pile)				Outer	;	φ1200	×	56.00	×	36Pile	
						Inner	;	φ1200	×	52.10	×	8Pile	
		Steel Pipe	Outer	Upper				t = 14 mm		(SKY490)			
				Lower				t = 14 mm		(SKY400)			
			Inner	---				t = 14 mm		(SKY400)			
	Calculation	Normal	δ		cm	0.04	≦	5.00	○	0.06	≦	5.00	○
			PNmax		KN/pile	2014	≦	4119	○	2015	≦	4119	○
			PNmin		KN/pile	1697	≧	-1959	○	1696	≧	-1959	○
		EQ	δ		cm	2.31	≦	5.00	○	2.24	≦	5.00	○
			PNmax		KN/pile	1928	≦	6178	○	1939	≦	6178	○
			PNmin		KN/pile	1674	≧	-3344	○	1681	≧	-3344	○
	Combined Stress (EQ)		SKY400		N/mm2	171.6	≦	210.0	○	183.0	≦	210.0	○
			SKY490		N/mm2	200.0	≦	277.5	○	209.4	≦	277.5	○

JICA Survey Team 14

2) Design Results at P11, 12



						Longitudinal				Transverse			
Column	Section	Cross Section			m	Oval	;	12.000	×	7.500			
		Re-bar	Main Re-bar	1st				D51	ctc	150			
				2nd				D51	ctc	150			
			Hoop	---				D22	ctc	150			
	Calculation	Earthquake (EQ)	σc		N/mm2	10.27	≦	15.00	○	8.99	≦	15.00	○
			σs		N/mm2	268.0	≦	300.0	○	206.9	≦	300.0	○
			τm		N/mm2	0.433	>	0.200	-	0.361	>	0.179	-
			Aw_req		mm2	1488.7	≦	3096.8	○	732.4	≦	2322.6	○
SPSP	Section	D(mm)×L(m)×nos(pile)				Outer	;	φ1200	×	60.00	×	40Pile	
						Inner	;	φ1200	×	55.10	×	8Pile	
		Steel Pipe	Outer	Upper				t = 16 mm		(SKY490)			
				Lower				t = 14 mm		(SKY400)			
			Inner	---				t = 14 mm		(SKY400)			
	Calculation	Normal	δ		cm	0.47	≦	5.00	○	0.07	≦	5.00	○
			PNmax		KN/pile	2745	≦	3746	○	2732	≦	3746	○
			PNmin		KN/pile	2396	≧	-1865	○	2391	≧	-1865	○
		EQ	δ		cm	2.65	≦	5.00	○	2.19	≦	5.00	○
			PNmax		KN/pile	2599	≦	5584	○	2610	≦	5584	○
			PNmin		KN/pile	2321	≧	-3092	○	2310	≧	-3092	○
	Combined Stress (EQ)		SKY400		N/mm2	139.3	≦	210.0	○	150.8	≦	210.0	○
			SKY490		N/mm2	240.6	≦	277.5	○	236.6	≦	277.5	○



JICA Survey Team 16

Cable Stayed Bridge

Detailed Design-2

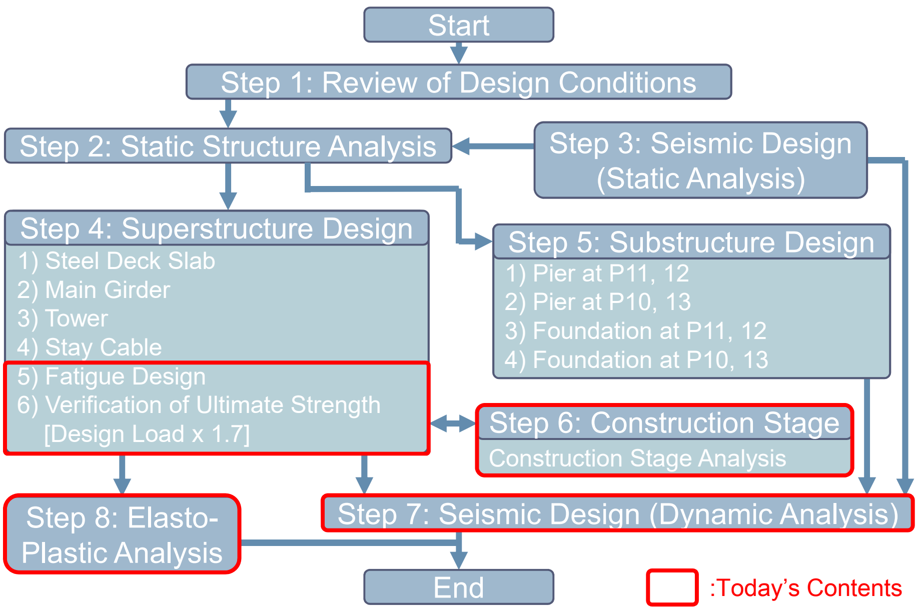
JICA Survey Team 17

Contents for Cable Stayed Bridge

1. Fatigue Design
2. Verification of Ultimate Strength
3. Construction Stage Analysis
4. Seismic Design (Dynamic Analysis)
5. Elasto-plastic Analysis

JICA Survey Team 18

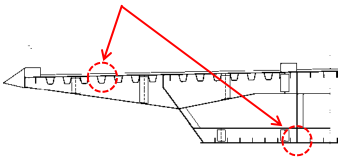
1. D/D Flow and Remaining Issues



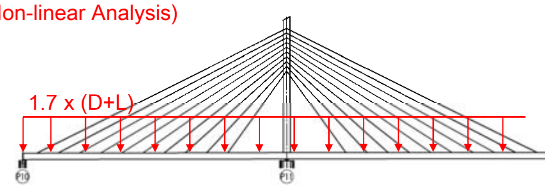
JICA Survey Team 19

- Description for Today's Contents -

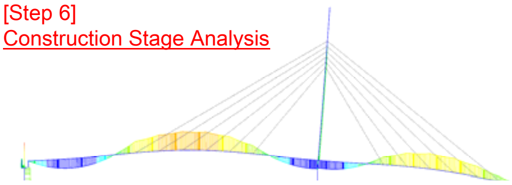
[Step 4-5]
Fatigue Design
(Steel Deck Panels)



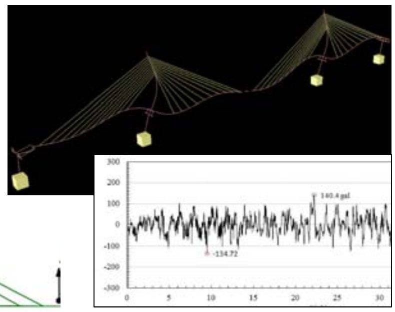
[Step 4-6]
Verification of Ultimate Strength
(Non-linear Analysis)



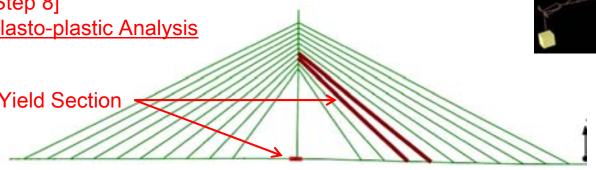
[Step 6]
Construction Stage Analysis



[Step 7]
Seismic Design [Dynamic Analysis]



[Step 8]
Elasto-plastic Analysis

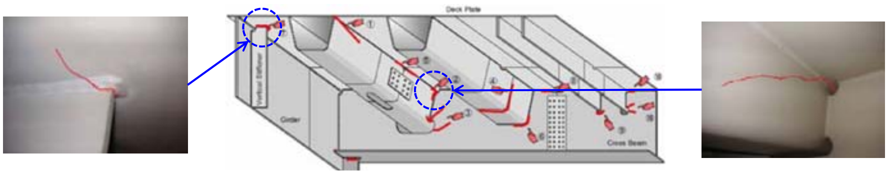


JICA Survey Team 20

2. Step4-5) Fatigue Design

1) Steel Deck Plate

- Due to a cyclic live load, "Fatigue Crack" was occurred at Steel Deck Slab in Japanese Road in the past.



- In order to prevent such cracks, rib arrangement and deck plate thickness are decided by "Fatigue Design Recommendations for Steel Structure (Japan Road Association)".
  ⇒ It can be considered that the Steel Deck Slab has enough durability for Fatigue.
  ⇒ Thickness of deck plate was decided by the Fatigue Design.

Deck Plate
t = 12mm→16mm

Item	Material	Plate Thickness	Load Case	Stress Resultants (σ or τ)	Allowable Value
Deck	SM400	16mm	D+ETR	45.60	140.00

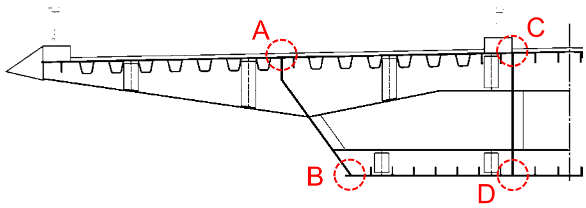
Stress is less than fatigue allowance

JICA Survey Team 21

2) Welding at Web and Flange

- Welding at web and flange should be checked by the Fatigue Design based on the Japanese Bridge Design Specifications.
- Compare girder stress and fatigue allowance (fatigue allowance depend on welding type and plate thickness).

Check Points



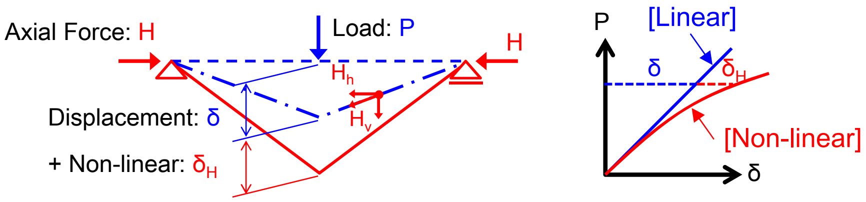
Section	Description of Welding	Stress	Allowable Value for Fatigue
A	Steel Deck and Outer Web	4 N/mm^2	109 N/mm^2
B	Bottom Flange and Outer Web	54 N/mm^2	92 N/mm^2
C	Steel Deck and Inner Web	2 N/mm^2	109 N/mm^2
D	Bottom Flange and Inner Web	54 N/mm^2	92 N/mm^2

JICA Survey Team 22

3. Step4-6) Verification of Ultimate Strength

1) Theory of Non-Linear Characteristics

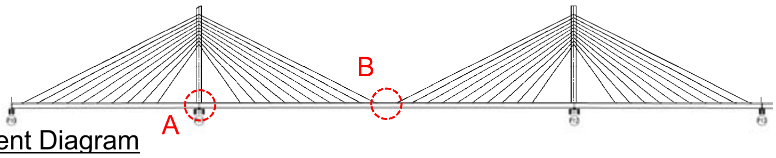
Non-linear Characteristics



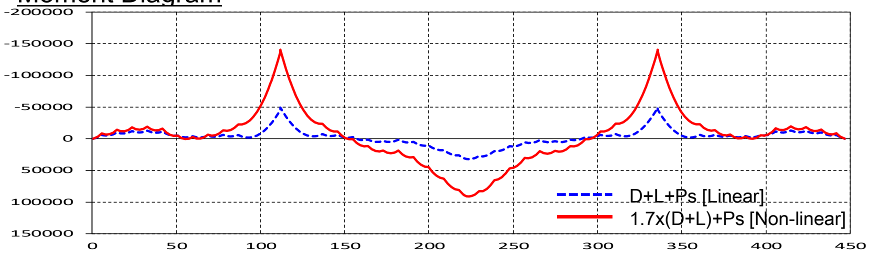
- In the Linear analysis, effect of axial force “H” is neglected at calculation for vertical displacement. However, in the Non-linear analysis, the axial force cause vertical displacement and displacement become slightly larger.
- Cable stayed bridge has axial force at girder and tower and shows Non-linear characteristics. Therefore, it should be studied whether the structure has necessary safety ratio (Steel:1.7) or not.

2) Ultimate Strength

- Apply 1.7x(D+L) load into the model and check stress resultants.



Moment Diagram



Maximum/Minimum Nominal Stress at Bottom Flange (SM490Y) (N/mm^2)

Section	D+L+Ps [Linear]	Allowable Value	1.7x(D+L)+Ps [Non-linear]	Allowable Value
A	-122	-158	-255	-269
B	71	210	177	357

(Difference of Linear and Non Linear is 0.5~1.0% in this project)

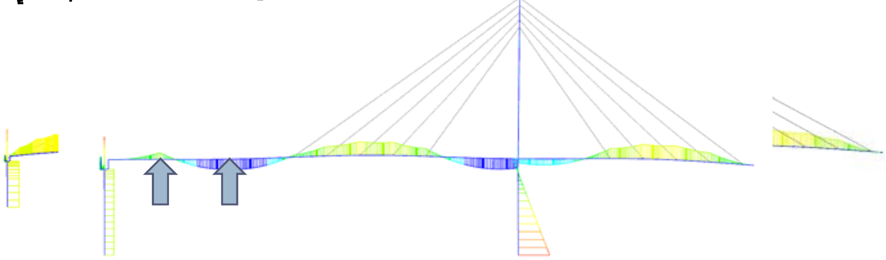
4. Step6) Construction Stage Analysis

1) Construction Step for Cable Stayed Bridge (Main Span)

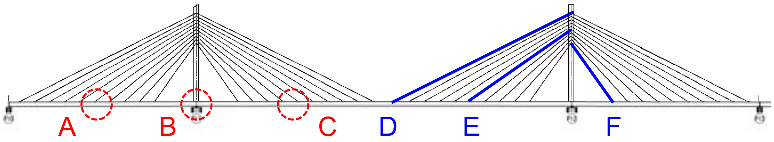
- Bent Erection Method(Side Span) + Cantilever Erection Method (Main Span) was proposed for Cable Stayed Bridge.
- Stress at girder and cable tension at each construction steps should be checked by Construction Stage Analysis.

Construction Steps

[Step4. 5th Cables]



2) Girder Stress and Cable Tension at Construction Stage



Maximum/Minimum Nominal Stress and Cable Tension

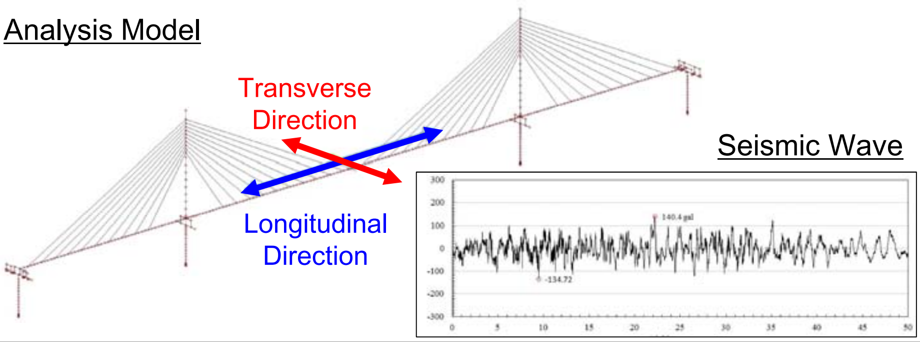
Section		Girder Stress/Cable Tension		Allowable Value
Girder [Max/Min Stress]	A	Max	35 N/mm^2	263 N/mm^2
		Min	-106 N/mm^2	-130 N/mm^2
	B	Max	39 N/mm^2	263 N/mm^2
		Min	-50 N/mm^2	-198 N/mm^2
	C	Max	5 N/mm^2	263 N/mm^2
		Min	-76 N/mm^2	-128 N/mm^2
Cable [Tension]	D	4359 kN		9135 kN
	E	3365 kN		4828 kN
	F	3110 kN		4828 kN

5. Step7) Seismic Design (Dynamic Analysis)

1) Dynamic Analysis

- Cable Stayed Bridge is complex structure, therefore safety performance should be verified by Dynamic Analysis.
- Apply standard seismic wave in Japanese Design Specifications. (Max: 140.4gai, Min:-134.72gal, Time: 50 sec)≒ (k_h=0.30)
- Analysis was conducted in longitudinal direction and transverse direction.

Analysis Model



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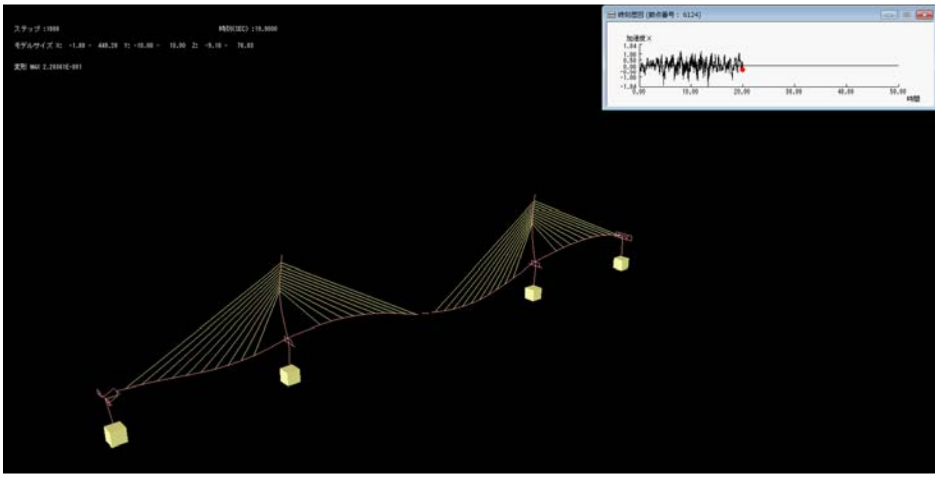
2) Movie (Longitudinal Direction)



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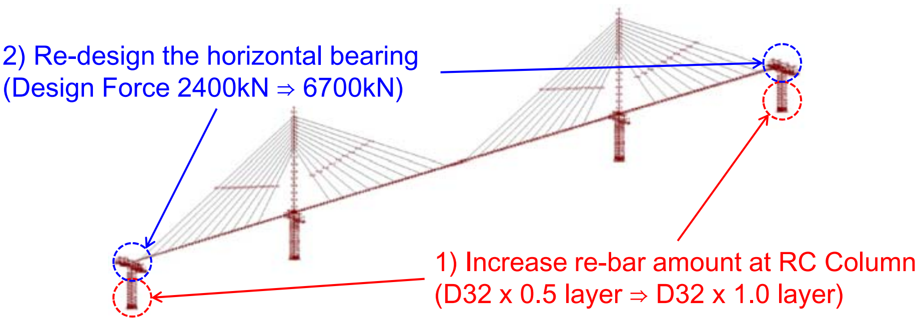
3) Dynamic Analysis Results

- Stress Resultants at Girder and Tower were less than the allowable value.
- Stress at RC column in transverse direction was over the allowable value. ⇒ 1) Increase re-bar amount
- Horizontal force (transverse) at horizontal bearing became bigger than the design force in static analysis
  ⇒ 2) Re-design the horizontal bearing



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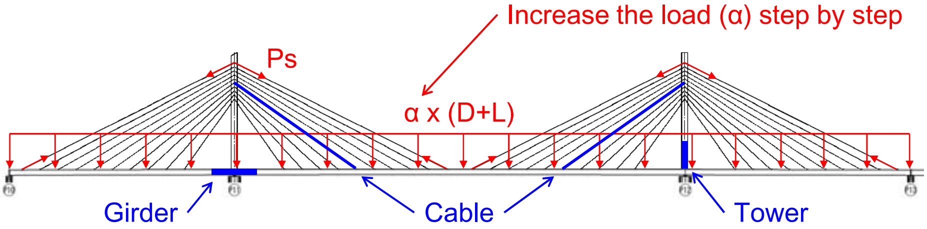
6. Step8) Elasto-plastic Analysis

1) Elasto-plastic Analysis

- Cable Stayed Bridge is complex structure and it is difficult to estimate ultimate strength and collapse mode.
- In order to verify safety performance of the structure at ultimate state, Elasto-plastic Analysis was conducted.
  (Plastic buckling of girder/tower and plastic behavior of cable can be evaluated appropriately by the elasto-plastic analysis.)

Analysis Method



Check the “α” when the structure is yielded

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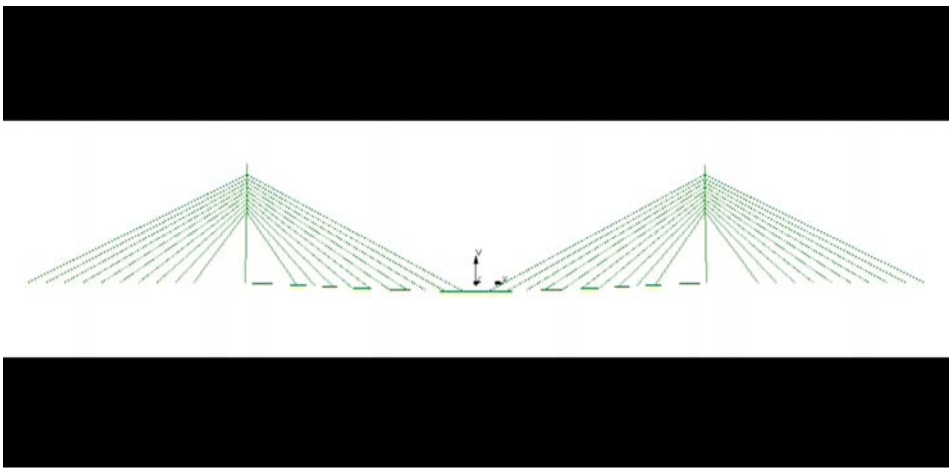
2) Movie (Yielding Step)

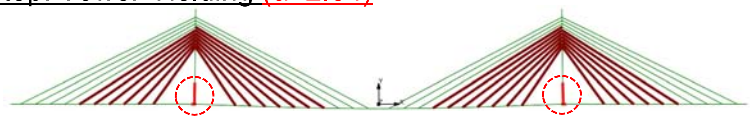


3) Analysis Results

1st Step: Girder Yielding (α=2.07)

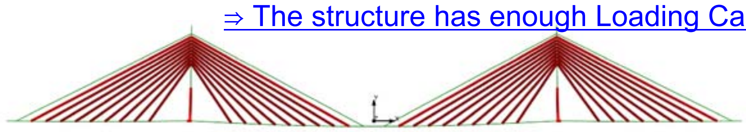
2nd Step: Cable Yielding (α=2.51)

3rd Step: Tower Yielding (α=2.84)

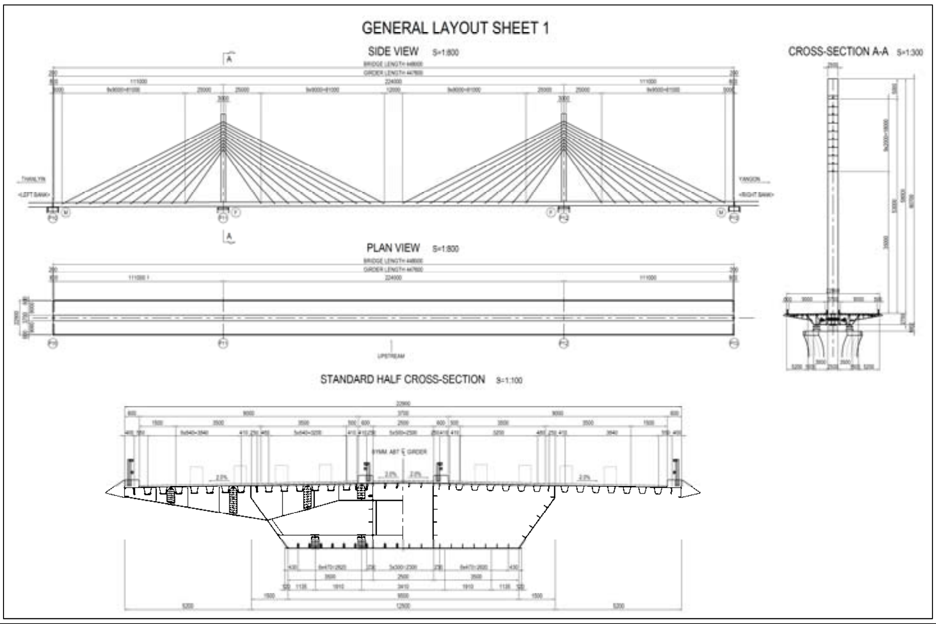


Ultimate State (α=2.98)

⇒ The structure has enough Loading Capacity

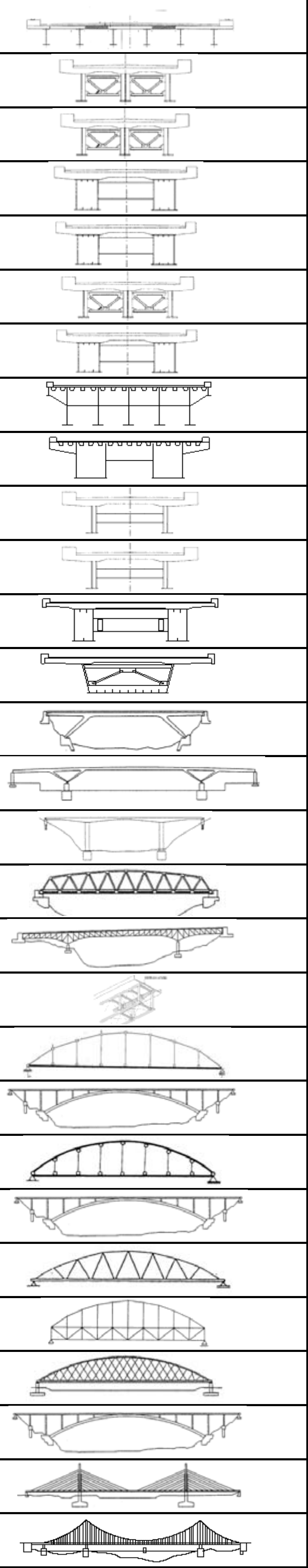


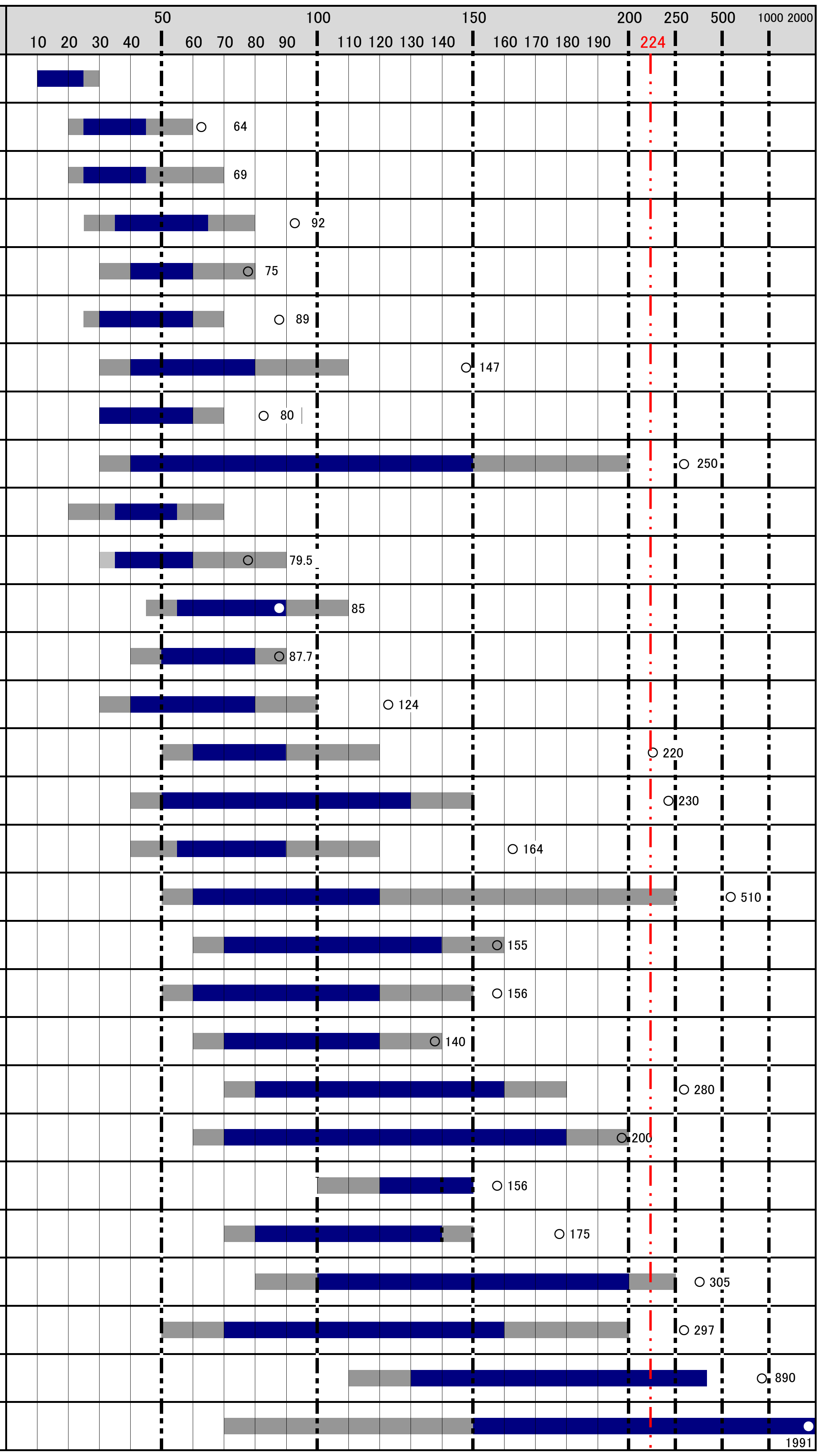
7. D/D Results of Cable Stayed Bridge



Appricable Span of Steel Bridge

Bridge Type		Typical Cross Section	Maximum Span in Japan (m)
Plate Girder Bridge	Simple Composite H-Girder		
	Simple Non-composite I-Girder		64
	Simple Composite I-Girder		69
	Simple Non-composite Box Girder		92
	Simple Composite Box Girder		75
	Continuous Non-composite I-Girder		89
	Continuous Non-composite Box Girder		147
	Steel Plate Deck I-Girder		80
	Steel Plate Deck Box Girder		250
	Simple I-Girder (PC/Composite Slab)		
	Continuous I-Girder (PC/Composite Slab)		79.5
	Narrow Box Girder (PC/Composite Slab)		85
	Open Box Girder		87.7
Rigid Frame	Rigid Frame (π-Shape)		124
	Rigid Frame (V-Shape)		220
	Continuous Rigid Frame		230
Truss	Simple Truss Bridge		164
	Continuous Truss Bridge (Gerber Truss)		510
	Truss Bridge (PC/Composite Slab)		155
Arch	Langer		156
	Deck Langer		140
	Lohse		280
	Deck Lohse		200
	Langered truss		156
	Trussed Langer		175
	Nielsen Lohse		305
	Unstiffened Arch		297
Cable Stayed Bridge			890
Suspension Bridge			1991





Ordinaly Appricable Renge — Appricable Range — ○ Maximum Span in Japan

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Detailed Practice_Cable-stayed Bridge

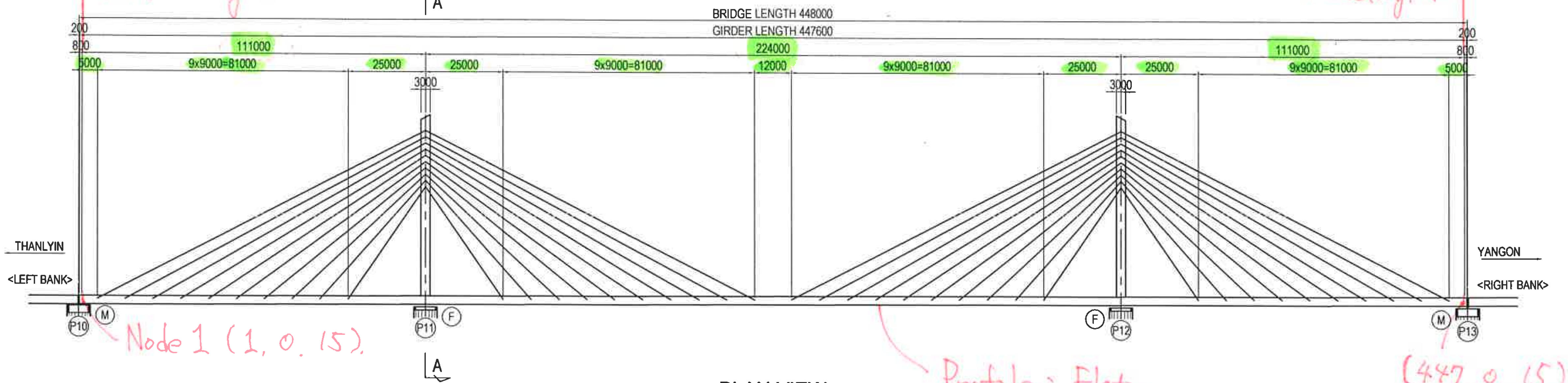
27th Oct 2017

1. Modeling

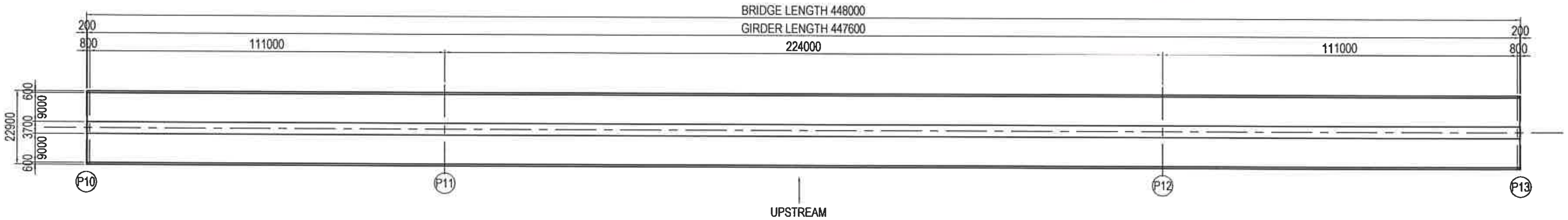
- Node
- Element
- Node Number: Girder: 1~ (1, 0, 15)
  Cable: 401~
  Tower: 501~, 601~
- Material: Girder, Tower, Cable
- Element: Girder, Tower: Beam
  Cable: Truss
- Element Number: Girder: 1~
  Cable: 401~
  Tower: 501~, 601~

GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (1)

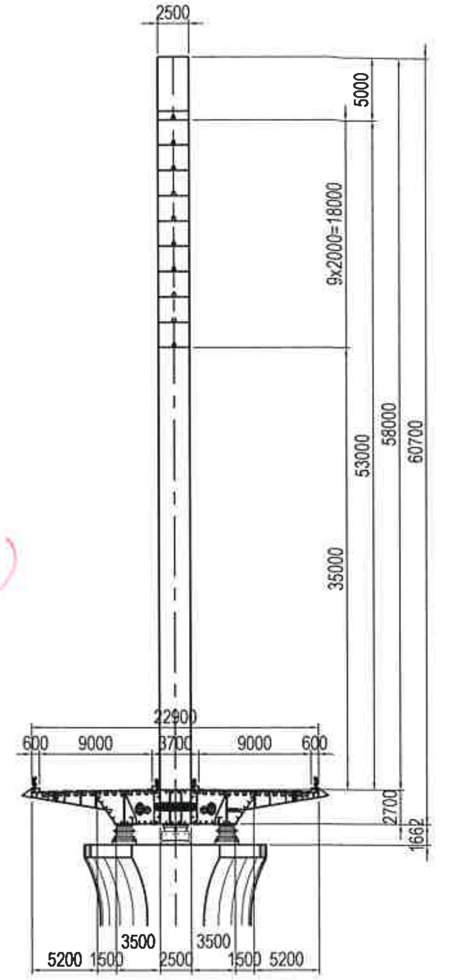
SIDE VIEW S=1:1600



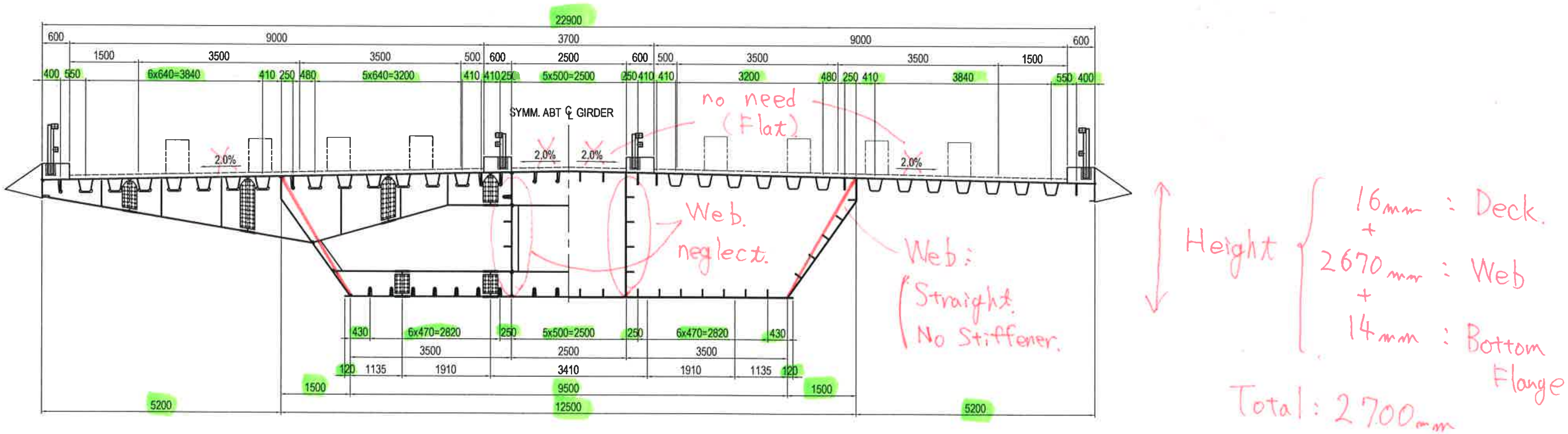
PLAN VIEW S=1:1600



CROSS-SECTION A-A S=1:600



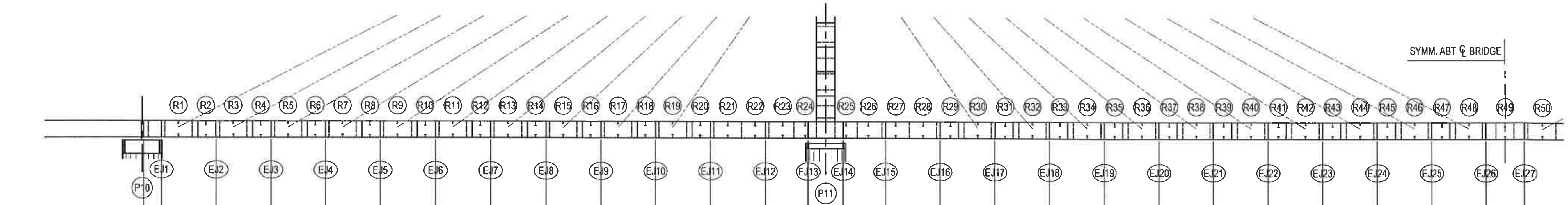
STANDARD HALF CROSS-SECTION S=1:100



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA			GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (1)	1
				CHECKED BY	T. HAYAKAWA				DWG No.
				APPROVED BY	Y. SANO				P1-CS-0005

GENERAL VIEW OF MAIN GIRDER (8)

ELEVATION N.T.S.



ERECTION SEGMENT NO.: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27

ERECTION SEGMENT LENGTH: 2990 | 11x9000=99000 | 6960 | 5700 | 6960 | 11x9000=99000 | 6380

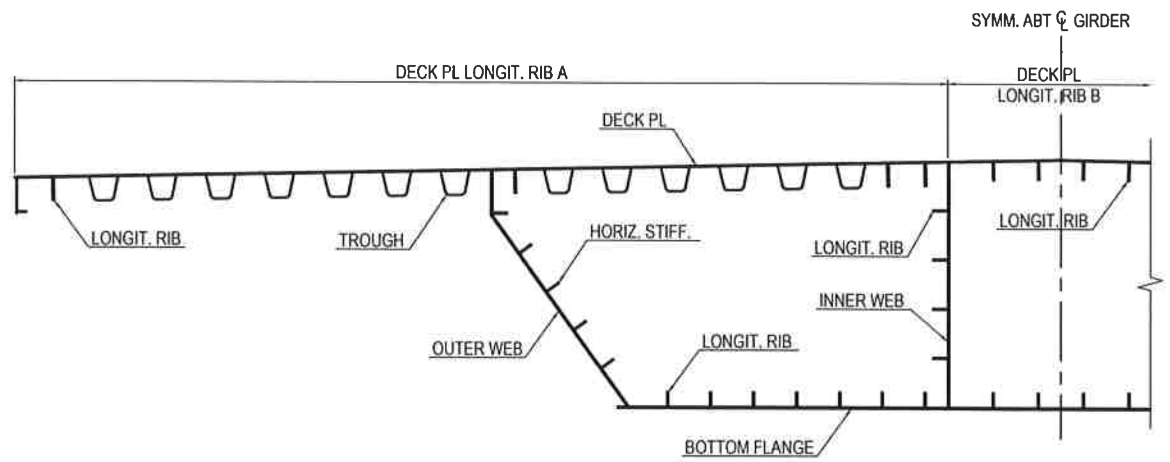
SECTION TYPE: TYPE-1 | TYPE-2 | TYPE-3 (A, B, C) | TYPE-4

UNIT: mm

		TYPE-1	TYPE-2	TYPE-3	TYPE-4
DECK PL	THK	16	16	16	16
	STEEL GRADE	SM400	SM400	SM400 / SM490Y / SM400	SM400
	TROUGH	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8	320 x 240 x 8
	LONGIT. RIB A	250 x 24	250 x 24	250 x 24	250 x 24
	LONGIT. RIB B	200 x 20	200 x 20	200x20	200 x 20
OUTER WEB	THK	14	14	17	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	HORIZ. STIFF.	160 x 16	160 x 16	200 x 20	160 x 16
INNER WEB	THK	14	14	18 / 35 / 18	14
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16
BOTTOM FLANGE	THK	14	11	15	11
	STEEL GRADE	SM490Y	SM490Y	SM490Y	SM490Y
	LONGIT. RIB	160 x 16	160 x 16	200 x 20	160 x 16

Modeling Section.

TYPICAL SECTION N.T.S.



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO.,LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.

	NAME	SIGNATURE	DATE
PREPARED BY	T.TOMODA	友田富雄	27. Nov.2017
CHECKED BY	T. HAYAKAWA	早川 知邦	28. Nov.2017
APPROVED BY	Y. SANO	佐野 ...	29. Nov.2017

DRAWING TITLE: GENERAL VIEW OF MAIN GIRDER (8)

PACKAGE: 1

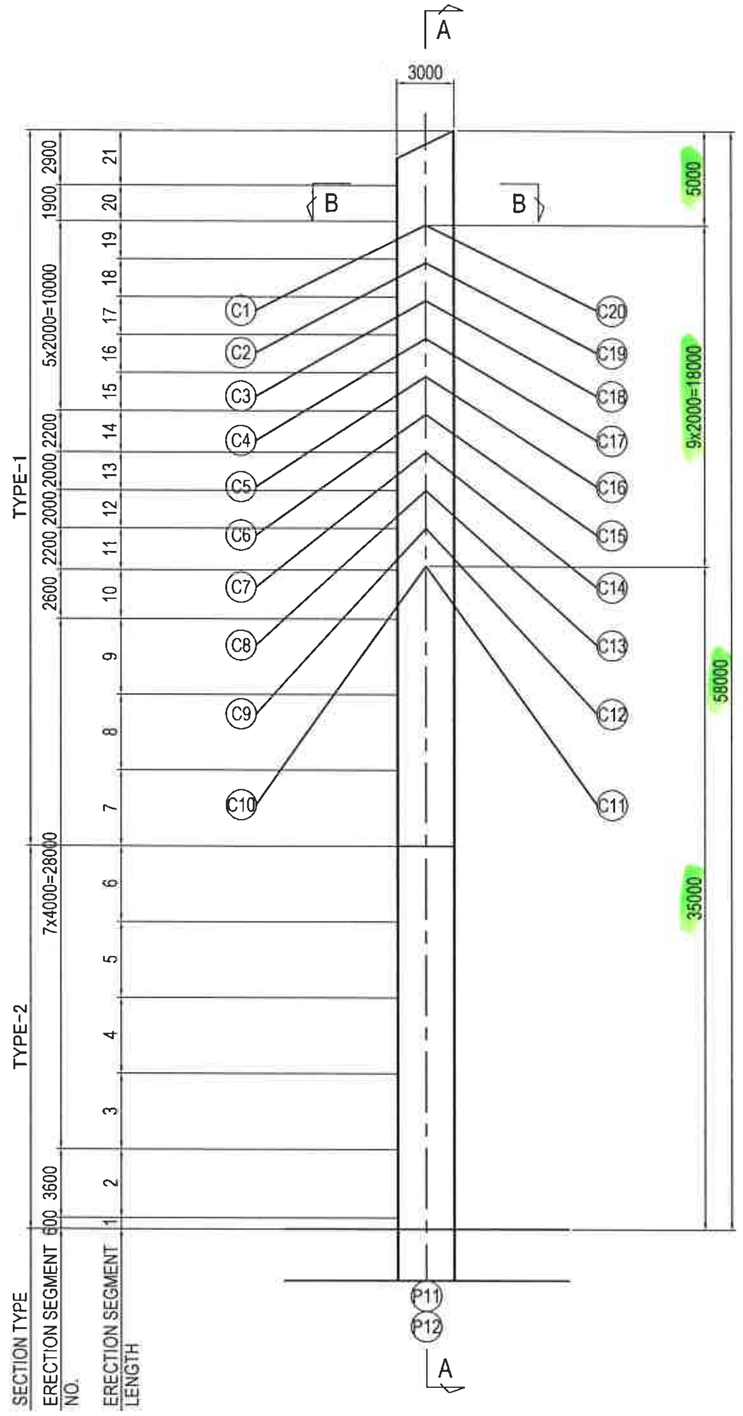
DWG No.: P1-CS-1008

GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (3)

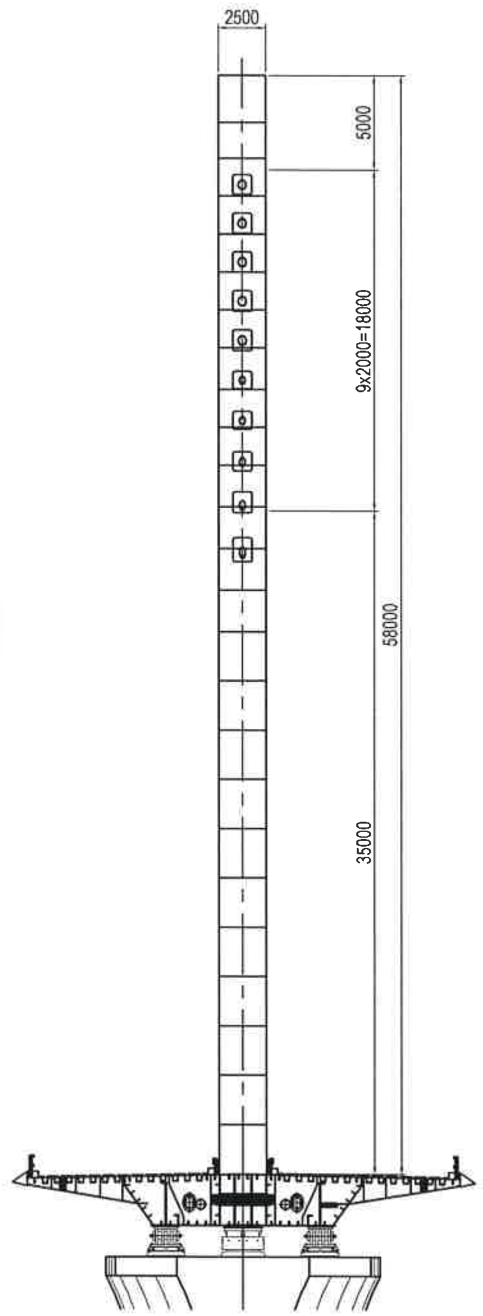
UNIT : mm

		SECTION TYPE		
		TYPE-1	TYPE-2	
FLANGE	STEEL GRADE	SM490Y	SM490Y
	THK	2420x40	2430x35
	LONGIT. RIB	2-280x27	2-260x25
WEB	STEEL GRADE	SM490Y	SM490Y
	THK	3000x40	3000x35
	LONGIT. RIB	5-250x25	5-250x25

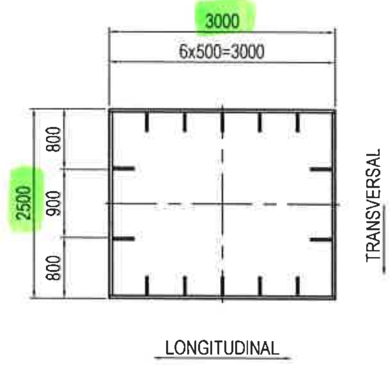
SIDE VIEW S=1:400



SECTION A-A



SECTION B-B



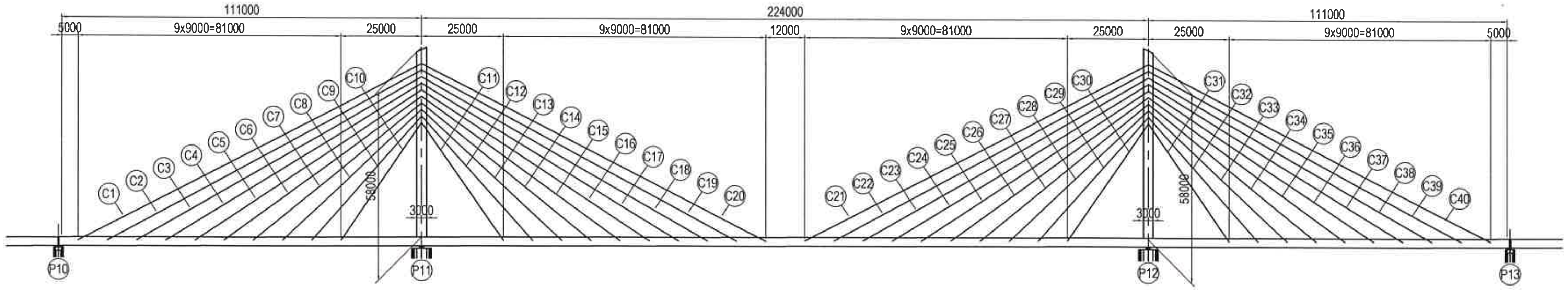
STAY CABLE NO.	NOS.	TYPE
C1	2	FUT-H (Φ15.6×70)
C2	2	
C3	2	
C4	2	
C5	2	
C6	2	FUT-H (Φ15.6×37)
C7	2	
C8	2	
C9	2	
C10	2	
C11	2	FUT-H (Φ15.6×37)
C12	2	
C13	2	
C14	2	
C15	2	
C16	2	FUT-H (Φ15.6×70)
C17	2	
C18	2	
C19	2	
C20	2	

Cable.

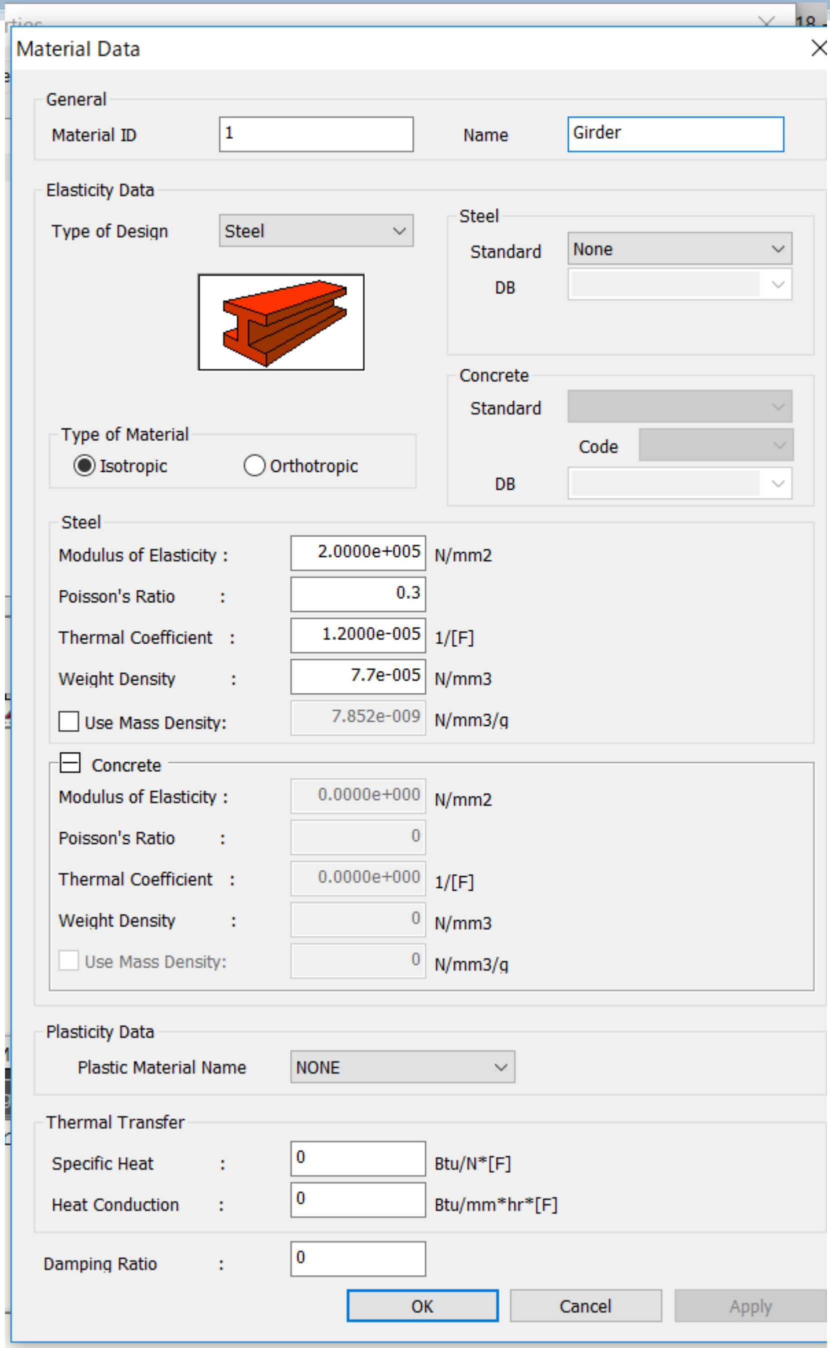
ANCHORAGE TYPE	37H	70H
CABLE CROSS SECTION	Φ180, 133, HDPE DUCT	Φ250, 190
NOM.AREA	5420 mm²	10255 mm²
TENSILE STRENGTH	9657 kN	18270 kN
ELASTIC MODULUS	190 kN/mm²	190 kN/mm²
UNIT WEIGHT (STRANDS+HDPE DUCT)	50.8 kg/m (37×1.268+3.1)	96.0 kg/m (70×1.268+5.6)

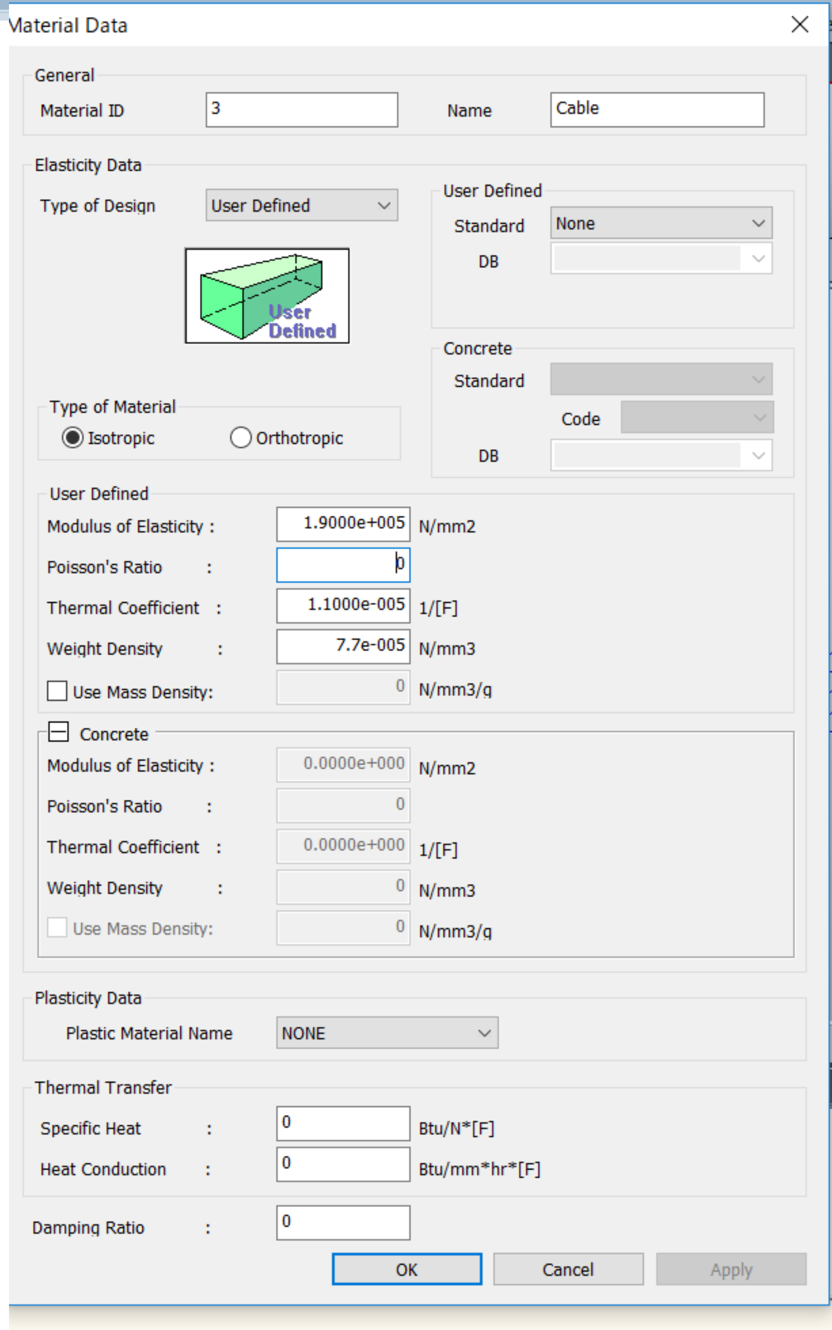
44H.
Area. 7178 mm²
Diameter 200 mm

LAYOUT DRAWING S=1:1600



PROJECT NAME	FINANCED BY	COUNTERPART	JICA STUDY TEAM		NAME	SIGNATURE	DATE	DRAWING TITLE	PACKAGE
DETAILED DESIGN ON BAGO RIVER BRIDGE CONSTRUCTION PROJECT	JAPAN INTERNATIONAL COOPERATION AGENCY	REPUBLIC OF THE UNION OF MYANMAR MINISTRY OF CONSTRUCTION DEPARTMENT OF BRIDGE	NIPPON KOEI CO., LTD. ORIENTAL CONSULTANTS GLOBAL CO., LTD. METROPOLITAN EXPRESSWAY COMPANY LIMITED CHODAI CO., LTD. NIPPON ENGINEERING CONSULTANTS CO.,LTD.	PREPARED BY	T.TOMODA			GENERAL VIEW OF STEEL CABLE STAYED BRIDGE (3)	1
				CHECKED BY	T. HAYAKAWA				DWG No.
				APPROVED BY	Y. SANO				P1-CS-0007


Material Data
General
Material ID 1
Name Girder
Elasticity Data
Type of Design Steel
Steel
Standard None
DB
Concrete
Standard
Code
DB
Type of Material
Isotropic
Orthotropic
Steel
Modulus of Elasticity : 2.0000e+005 N/mm2
Poisson's Ratio : 0.3
Thermal Coefficient : 1.2000e-005 1/[F]
Weight Density : 7.7e-005 N/mm3
Use Mass Density: 7.852e-009 N/mm3/g
Concrete
Modulus of Elasticity : 0.0000e+000 N/mm2
Poisson's Ratio : 0
Thermal Coefficient : 0.0000e+000 1/[F]
Weight Density : 0 N/mm3
Use Mass Density: 0 N/mm3/g
Plasticity Data
Plastic Material Name NONE
Thermal Transfer
Specific Heat : 0 Btu/N*[F]
Heat Conduction : 0 Btu/mm*hr*[F]
Damping Ratio : 0
OK
Cancel
Apply


Material Data
General
Material ID 3
Name Cable
Elasticity Data
Type of Design User Defined
User Defined
User Defined
Standard None
DB
Concrete
Standard
Code
DB
Type of Material
Isotropic
Orthotropic
User Defined
Modulus of Elasticity : 1.9000e+005 N/mm2
Poisson's Ratio : 0
Thermal Coefficient : 1.1000e-005 1/[F]
Weight Density : 7.7e-005 N/mm3
Use Mass Density: 0 N/mm3/g
Concrete
Modulus of Elasticity : 0.0000e+000 N/mm2
Poisson's Ratio : 0
Thermal Coefficient : 0.0000e+000 1/[F]
Weight Density : 0 N/mm3
Use Mass Density: 0 N/mm3/g
Plasticity Data
Plastic Material Name NONE
Thermal Transfer
Specific Heat : 0 Btu/N*[F]
Heat Conduction : 0 Btu/mm*hr*[F]
Damping Ratio : 0
OK
Cancel
Apply

JICA Study Team 5

1. Modeling

- Section: Girder: Steel Girder_Box
  Tower: DB_Box with Stiffener
  Cable: Value_Solid Round
- Assign Section: Girder, Tower, Cable
- Assign Material: Girder, Tower, Cable
- Boundary: P10, 13: Move (y,z,rx fix)
  P11, 12: Fix (x,y,z,rx fix)

JICA Study Team 6

2. Loading

- Dead Load: [Before Closing]: Da
  Girder + Cable Anchor
  Tower + Cable Anchor
  Fairing, Attachments,
  Inspection rail, Cable
  [After Closing]: Db
  Pavement, Curb, Median,
  Railing, Over Lay

⇒ Affect to Cable Pre-stressing

2. Loading

- Cable Pre-stress: PS
  1) Smoothen girder moment at completion state
     ⇒ Check at [Da + Db + PS]
  2) Bending moment at Tower ≒ 0
     at completion state
     ⇒ Check at [Da + Db + PS]
  3) Bending Moment at Joint ≒ 0 at girder closing
     ⇒ Check at [Da + PS]

- Dead Load

Items	Unit Weight		Width	Length	Nos	Total Load (kN/m)
[Before Closing]						
Girder	105.00	kN/m	---	---	---	
Girder Cable Anchor	35.28	kN/nos.	---	81	10	
Tower	43.00	kN/m	---	---	---	
Tower Cable Anchor	50.96	kN/nos.	---	18	10	
Fairing	5.00	kN/m	---	---	2	
Attachments	6.00	kN/m	---	---	1	
Inspection Rail	1.50	kN/m	---	---	2	
Cable	Refer "Cable Weight"					
[After Closing]						
Pavement	22.50	kN/m3	9	0.08	2	
Curb	24.50	kN/m3	0.6	0.336	2	
Median_Curb	24.50	kN/m3	0.6	0.324	2	
Median_Concrete Pavement	23.00	kN/m3	2.5	0.08	---	
railing	0.60	kN/m	---	---	2	
Over Lay	0.70	kN/m2	22.9	---	---	

- Cable Weight

Elem. No	Node No.		Type	Total Cable Weight				
				Cable	Girder Anchor	Tower Anchor	Girder Total	Tower Total
	Girder	Tower		(kN)	(kN)	(kN)	(kN)	(kN)
401	2	510	49	36.7	2.3	2.1	39.0	38.8
402	3	509	49	33.9	2.3	2.1	36.2	36.0
403	4	508	49	31.2	2.3	2.1	33.5	33.3
404	5	507	49	28.4	2.3	2.1	30.7	30.5
405	6	506	49	25.8	2.3	2.1	28.1	27.9

3. Analysis

- Check moment of girder by [Da+Db]
- Input cable force at [Pre-stress]
- Check moment of girder by [Da+Db+PS]
- Check force at Truss by [Da+Db+PS], [PS]

Cable seems to resist to the Pre-stress force

⇒ Change section property by "Section Manager"

JICA Study Team 11

4. Change Cable Property

- Make boundary group for Pre-stress
- Make boundary for Pre-stress
- Use “Section Manager”
  - Cable dose no resist to the Pre-stress force
- Boundary Change Assignment
  - Pre-stress: used as PS
  - Other Load: Normal
- Re-calculate
- Check [Pre-stress]
- Check [Da+Db+PS]

5. Live Load

- Moving Load Coad: AASHTO LRFD
- Traffic Line Lanes: Lane 1~6
  1) Eccentricity: Lane1: -9.35m
     Lane2~6: Pls Calculate
  2) Wheel Spacing: 1.8m
  3) Vehicular Load Distribution: Lane Element
  4) Moving Direction: Both
  5) Selected by 2 points (All Section of Girder)

5. Live Load

- Vehicles
  1) HL-93 TDM
  2) HL-93 TRK
- Vehicle Class
  1) HL-93: Select HL-93TDM and HL-93TRK
- Moving Load Cases
  1) Load Case: HL-93, Truck, Tandem
  2) Loading Effect: Combined
  3) Add - Vehicle Class: HL-93
     Min Lane: 0 Max Lane: 6
     Select All Lanes

5. Live Load

- Impact
  Load Combination "L-impact",
  HL-93, factor = 1.073
  Impact for Girder: i = 20/(50+L) [JSHB]
  Impact for Tower: i = 0.15 [from other projects]
  Impact for Cable: i = 0.2 [from other projects]

* In actually, we need to make 5 models
  impact for side span, impact for main span,
  impact for intermediate pier, impact for cable
  impact for tower

6. Temperature Effect

- Temperature Range
  10° ~ 40° (25° ±15°)
  Load Case: Temp+15, Temp-15
  Temp./Pre-stress, Element Temp.
  (Add to all of element)
- Load Combination
  Make "Temp", [Type: Envelope]
  Select "Temp+15" and "Temp-15"

JICA Study Team 15

7. Wind Load

- Wind Speed
  JSHB: U10 (10 minutes Average Wind Speed)
  MNBC: 3 sec Gust Wind Speed
  ⇒ Convert from 3 sec gust wind speed to U10
- Wind Load
  Based on JSHB, Guideline for Wind Resistance Design and other project.
  (1) Wind Load for Girder and Tower [kN/m]
  ⇒ Input by Element Beam Load
  (2) Wind Load for Cable [kN]
  ⇒ Input by Nodal Load

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7. Wind Load

- Wind Load for Live Load: w
  Ud: 32.3m/s (same with Girder)
  Cd: 1.6 (from JSHB)
  G: 1.9 (from JSHB)
  pd = ½ x ρ x Ud^2 x Cd x G [kN/m^2]
  w = ½ x pd x 1.0 m^2/m [kN/m]
- Loading Height
  From loading position to Bridge Surface: 1.5m
  Pavement: 0.08m
  From Deck to Center of G: 0.75m
  Total Height: m ⇒ m

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7. Wind Load

- Wind Load for Girder and Tower
  Element Beam Load
  [Global Y direction]
- Wind Load for Cable
  Nodal Load
  [FY direction]
- Wind Load for Live Load
  Element Beam Load
  [Global Y direction, Eccen. Global Z 2.5m]

8. Load Combination

- Load Combination at JSHB

  1) D+L 2) D+L+T 3) D+Wg 4) D+L+Wg

  5) D+Wt 6) D+L+Wt 7) D+Wg+T

  8) D+L+Wg+T 9)D+Wt+T 10) D+L+Wt+T

Table 3.1.1 Increase factor of allowable stress

	Combination of loads	Increase coefficient
1)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*)	1.00
2)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*) + Effects of temperature changes (*T*)	1.15
3)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*) + Wind load (*W*)	1.25
4)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*) + Effects of temperature changes (*T*) + Wind load (*W*)	1.35
5)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*) + Braking load (*BK*)	1.25
6)	Principal loads (*P*) + Special loads corresponding to "principal load" (*PP*) + Collision load (*CO*)	
	For a steel member	1.70
	For a reinforced concrete member	1.50
7)	Principal loads other than live load and impact + Effects of earthquakes (*EQ*)	1.50
8)	Wind load (*W*)	1.20
9)	Braking load (*BK*)	1.20
10)	Erection load (*ER*)	1.25

Items		Unit Weight		Width	Length	Nos	Total Load (kN/m)
[Before Closing]							
	Girder	105.00	kN/m	---	---	---	
	Girder Cable Anchor	35.28	kN/nos.	---	81	10	
	Tower	43.00	kN/m	---	---	---	
	Tower Cable Anchor	50.96	kN/nos.	---	18	10	
	Fairing	5.00	kN/m	---	---	2	
	Attachments	6.00	kN/m	---	---	1	
	Inspection Rail	1.50	kN/m	---	---	2	
	Cable	Refer "Cable Weight"					
[After Closing]							
	Pavement	22.50	kN/m3	9	0.08	2	
	Curb	24.50	kN/m3	0.6	0.336	2	
	Median_Curb	24.50	kN/m3	0.6	0.324	2	
	Median_Concrete Pavement	23.00	kN/m3	2.5	0.08	1	
	Railing	0.60	kN/m	---	---	4	
	Over Lay	0.70	kN/m2	22.9	---	---	

Elem. No	Node No.		Type	Cable	Coordinates						Cable Length	Cable Weight	Total Cable Weight				
				Unit Weight	Girder			Tower			(m)	(m)	Cable	Girder Anchor	Tower Anchor	Girder Total	Tower Total
	Girder	Tower		(kN/m)	x	y	z	x	y	z			(kN)	(kN)	(kN)	(kN)	(kN)
401	2	510	49	0.61838	6	0	15	112	0	68	118.51	73.3	36.7	2.3	2.1	39.0	38.8

Cable

Strand Nos.	Area	Yield	Tensile	HDPE D	Weight
ϕ15.6×n	mm2	kN	KN	mm	kg/m
1	146.5	222	261	−	1.288
19	2783	4218	4959	140	24.5
24	3516	5328	6264	160	30.9
31	4541	6882	8091	160	39.9
37	5420	8214	9657	180	47.7
44	6446	9768	11484	200	56.7
49	7178	10878	12789	200	63.1
55	8057	12210	14355	225	70.8
61	8936	13542	15921	225	78.6
64	9376	14208	16704	250	82.4
70	10255	15540	18270	250	90.2
77	11280	17094	20097	250	99.2
85	12452	18870	22185	280	109.5
91	13331	20202	23751	280	117.2
96	14064	21312	25056	315	123.6
111	16261	24642	28971	315	143
127	18605	28194	33147	315	163.6

Fix_Anchor: Tower

Strand	Weight
ϕ15.6×	kg
19	95
24	121
31	139
37	165
44	197
49	215
55	227
61	260
64	265
70	304
77	328
85	347
91	390
96	413
102	
105	
111	492
121	
127	586

Adjust Anchor: Gi

Strand	Weight
ϕ15.6×	kg
19	88
24	113
31	140
37	170
44	213
49	238
55	261
61	303
64	328
70	368
77	411
85	446
91	502
96	521
102	
105	
111	637
121	
127	755

[Basic Wind Speed]

Wind Speed U10 30 m/s

[Basic Height]

Girder

Ave. Height 15 m

Tower 58 m

65% of the tower 37.7 m

Total Height 52.7 m

Tower Section (m)	
Longi.	Trans.
3	2.5

Cable

Top of Tower 73 m

Ave. of Girder 15 m

Height for Cable 44 m

[Basic Design Wind Speed]

Ud = μ1 x U10

μ1 = (Z/10)^(1/8)

Items	Basic Height	μ1	U10 (m/s)	Ud (m/s)
Girder	15	1.052	30	31.6
Tower	52.7	1.231	30	36.9
Cable	44	1.203	30	36.1

[Total Height of Girder]

Railing	0.40	m			
Curb	0.25	m			
Pavement	0.08	m			
Girder	2.70	m			
Total: D	3.43	m	⇒	3.50	m

[Loading Height of Wind]

Total of Girder/2	1.75	m			
Girder	-2.70	m			
From CoG to Deck	0.75	m			
Total	-0.20	m	⇒	0.00	m

[Total Width of Girder]

Width: B 22.9 m

[Drag Coefficient]

B/D 6.54

CD for Girder	1≦B/D≦8	2.1-0.1(B/D)
	8<B/D	1.3

Items	CD

Girder	1.45	
Tower	1.80	from other projects
Cable	0.70	from other projects

[Projected Area]

Items	Area (m2)
Girder	1561.0
Tower	173.0

[Modification Factor]

Design Object	Girder	Tower	
Direction	Trans.	Trans.	
Girder μ2	1.90	1.65	from other projects
Tower μ3	1.90	1.65	from other projects
Cable μ2	1.90	1.65	from other projects

[Wind Load] (kN/m2)

(From Other Project)

Girder, Cable PD = 1/2・ρ・Ud^2・μ2・CD

Tower PD = 1/2・ρ・Ud^2・μ3・CD

ρ：Air Density(1.23kg/m3)

Design Object	Girder	Tower
Direction	Trans.	Trans.
Girder	1.68	1.46
Tower	2.87	2.49
Cable	1.07	0.93

[Wind Load] (kN/m)

(From Other Project)

Girder, Cable PD = 1/2・ρ・Ud^2・μ2・CD・h

Tower PD = 1/2・ρ・Ud^2・μ3・CD・h

ρ：Air Density(1.23kg/m3)

Design Object		Girder	Tower
Direction		Trans.	Trans.
Girder	h	3.50	3.50
	PD	**5.90**	**5.10**
Tower	h	3.00	3.00
	PD	**8.60**	**7.50**

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Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Detailed Practice_Cable-stayed Bridge

15th Jan 2018

JICA Study Team 2

1. Parametric Analysis

- In order to decide the best structure type and grasp the characteristics of cable-stayed bridge, parametric analysis should be conducted.
- Items to be studied;
  (1) Girder Rigidity
  (2) Tower Height
  (3) Cable Arrangement
  (4) Cable Number
- Load Case: Dead Load + Pre-stress
  Dead Load + Pre-stress + Live Load

1. Parametric Analysis

(1) Girder Rigidity
Change Girder Rigidity
(Height, Dimension, Value, etc.)

(2) Tower Height
Change Tower Height (Low, Middle, High, etc.)

(3) Cable Arrangement
Fan Type, Semi-fan Type, Radial Type

(4) Cable Number
Change Cable Number

1. Parametric Analysis

- Analysis Case
  Please try to analyze 3~5 cases for each Items
- Check Points
  - Displacement (D+PS, D+PS+L)
  - Stress Resultants (N, M, Q) and Diagram
  - Reaction Force at Bearing, etc.
- Report
  - Prepare "Conclusion Report" (ppt, excel, word)
- Period: 15th of Jan. ~ 26th of Jan.

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Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Detailed Practice_Cable-stayed Bridge

26th Jan 2018

JICA Study Team 2

1. Section Design

- In order to decide the detail of section (Plate thickness, Cable size, etc.), stress, displacement, tension force should be checked.
- Items to be studied;

(1) Load Combination (Increase Coefficient)
(2) Allowable Stress (Axial, Bending, Shear)
(3) Allowable Cable Tension (Safety Factor=2.5)
(4) Stress Resultants ⇒ Stress
(5) Stress Check based on JSHB
(6) Displacement Check under Live Load

1. Section Design_(1) Load Combination

- Load Combination and Increase Coefficient
  - "Chapter 2 Loads"
  - Load Combination: 1) DL+PS+LL
    2) DL+PS+LL+T
- Increase Coefficient
  - "Chapter3 Allowable Stress" Table3.1.1

1. Section Design_(2)(3) Allowable Value

- Calculation of Allowable Stress
  - [Tensile]
    Axial Stress, Bending Stress : Table 3.2.1
  - [Compressive]
    Axial Stress : Table 3.2.2
    Bending Stress : Table 3.2.3
  - Shear Stress: Table 3.2.4
- Calculation of Allowable Cable Tension
  - Safety Factor: Table 16.5.1

JICA Study Team 5

1. Section Design_(2)(3) Allowable Value

- Effective Buckling Length
  - Girder: Distance between 2 cables or distance from cable to support point
  - Tower: In-plane: 0.7h
    Out-plane: 1.0h

JICA Study Team 6

1. Section Design_(4) Stress

- Calculation of Stress
  - Axial Force ⇒ Axial Stress (σ = N/A)
  - Bending Moment ⇒ Bending Stress (σ = M/I x y)
  - Shear Force ⇒ Shear Stress ($\tau = Q/A_{Web}$)

  (Torsion, Shear Stress of Flange, Effective Width: Neglect)
- Stress Resultants
  - Use only 3 components
    (Axial, Moment at Y axis, Shear in Z direction)

1. Section Design_(5) Stress Check

- Check by Each Stress
  - Axial Stress vs Allowable Axial Stress
  - Bending Stress vs Allowable Bending Stress
  - Shear Stress vs Allowable Shear Stress
- Check by Combination of Stress
  - Combination of Axial and Bending
    Refer “Chapter 4.3”
  - Combination of Bending and Shear
    Refer “Chapter 10.2.5”

1. Section Design_(6) Displacement Check

- Displacement Check under Live Load w/o impact
  - Refer “Chapter 2.3”

JICA Study Team 9

2. Others

- Design Points:

  At least, please try to calculate 3 points for Girder, 2 points for tower and 2 cables.
- Stress:

  For Flange: Axial and Bending Stress

  For Web: Axial, Bending and Shear Stress

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Detailed Practice_Cable-stayed Bridge

8th Feb 2018

1. Seismic Design by JSHB

- Seismic design of bridges based on JSHB is relation of;

Seismic Level for Design [2 levels]
x
Importance Category of Bridge [2 Categories]
↓
Required Performance [3 performances]

1. Seismic Design by JSHB

1) Seismic Level for Design

[Level 1 Earthquake]

Corresponds to an earthquake with high probability of occurrence during the bridge service life (100 years).

[Level 2 Earthquake]

Corresponds to an earthquake with less probability of occurrence during the bridge service life.

In the Level 2, there are 2 types of seismic wave;

Type1: Plate boundary type

Type2: Inland direct strike type

1. Seismic Design by JSHB

2) Importance Category of Bridge

[Class A]

The bridges of standard importance.

(Other than the Class B)

[Class B]

The bridges of high importance.

(National Expressways, Urban Highways, Highway Viaducts, Rail/Road Over Bridges, etc.)

1. Seismic Design by JSHB

3) Required Seismic Performance

[Performance Level1]

Bridge keeping its sound functions during earthquake.

[Performance Level 2]

Bridge sustaining limited damages during an earthquake and capable of recovery within a short period.

[Performance Level 3]

Bridge sustaining no critical damages during an earthquake

Table C-2.2.1 Seismic Performance of Bridges

Seismic Performance	Seismic Safety Design	Seismic Serviceability Design	Seismic Repairability Design	
			Emergency Repairability	Permanent Repairability
Seismic Performance Level 1: Keeping the sound functions of bridges	To ensure the safety against girder unseating	To ensure the normal functions of bridges	No repair work is needed to recover the functions	Only easy repair works are needed
Seismic Performance Level 2: Limited damages and recovery	Same as above	Capable of recovering functions within a short period after the event	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair works
Seismic Performance Level 3: No critical damages	Same as above	—	—	—

⇒ Refer Chapter 5

1. Seismic Design by JSHB

4) Relation of these items

Seismic Level		Class A Bridges	Class B Bridges
Level 1 Earthquake		Seismic Performance 1	
Level2 Earthquake	Type 1	Seismic Performance 3	Seismic Performance 2
	Type 2		

2. Verification of Seismic Performance

1) Design Steps

[Level 1 Earthquake]

- Static Analysis or Dynamic Analysis
- Compare with Allowable stress

↓

[Level 2 Earthquake]

- Static Analysis or Dynamic Analysis
- Compare with Ultimate Strength or Displacement
- Consider plastic behavior

JICA Study Team 9

2. Verification of Seismic Performance

2) Static Analysis or Dynamic Analysis

- Depend on seismic performance
- Depend on bridge type

3) Static Analysis Method

- Design Force

  H = kh x W (kh: Horizontal Seismic Coefficient)
- Apply the force into longi. and trans. direction
- kh: depend on the period, soil type, area

JICA Study Team 10

2. Verification of Seismic Performance

4) Dynamic Analysis Method

- Input design seismic wave
- Seismic wave: Actual recorded wave
- Depend on soil type

JICA Study Team 11

3. Design Seismic Load

1) Horizontal Force
   - kh = 0.3
   - Horizontal Force H = 0.3 x W
   - W: All dead load
     Girder and Tower: Element Beam Load
     Cable: Nodal Load

2) Load Case
   - Seismic Load for Longitudinal Direction
   - Seismic Load for Transverse Direction

3) Load Combination
   - D + PS + EQ (Increase Factor: 1.5)

JICA Study Team 12

4. Difference of Support Conditions

1) M-F-F-M
   Move - Fix - Fix -Move

2) M-F-M-M
   Move - Fix - Move - Move

3) E-E-E-E
   Elastic - Elastic - Elastic - Elastic
   ※Point Spring Support: SDx = 10000kN/m
   SDy = 10000kN/m
   SDz = 4000000kN/m

Check Reaction Force and Displacement under Temperature Load and Seismic Load

1

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Detailed Practice_Cable-stayed Bridge

14th Feb 2018

JICA Study Team 2

1. Design Exercise

Design Conditions 1

Bridge Type: 3-span cable-stayed Bridge

Span Length: 150m + 300m + 150m

Structure: Steel Girder, Steel Tower, Cable

Girder and Tower Section:

Basic shape is same with previous model, but you can change shape, height, etc. as you want.

(Top Flange: 16mm)

Cable Section: Decide by yourself

1. Design Exercise

Design Conditions 2

Tower Height: Decide by yourself
(1:2, L/6 ~ L/4, etc.)

Cable Number: Decide by yourself

Cable Arrangement: Select different arrangement
(Harp or Semi-fan)

Pre-stress: Decide by yourself

Boundary Conditions: M-F-F-M

1. Design Exercise

Design Conditions 3

Design Load : Dead Load, Pre-stress
Live Load with impact
Temperature Effect (25±15℃)
~~(Wind and Seismic)~~

(Girder and Tower: DL = 1.4 x Area x Density)
(Cable Anchor: Same with previous model)
(Other: Same with previous model)

Load Case: DL+PS+LL, DL+PS+LL+T
~~(DL+PS+LL+W, DL+PS+EQ)~~

1. Design Exercise

Design Conditions 4

Section Type: Side Span 2~3 Type
Center Span 2~3 Type
Tower 1~2 Type

Effective Buckling Length
Girder: Cable or Support Distance
Tower: 1.0h or 0.7h

Check Items: σb, σa, τ, σb+σa, σb+τ, LL_disp

1. Design Exercise

Design Conditions 5

Conclusion:

- Design Conditions
- Diagram: DL, DL+PS, DL+PS+LL
- Section Summary of Girder, Tower, Cable
- Section Calculation Results

~~(- Comparison of structure type)~~

Prepare design report (Word, Excel, etc.) including these contents.