TUGAS AKHIR - RC18-4803

MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM *EXTRADOSED*

MUHAMMAD ANHAR PRAKOSO
NRP. 03111640000084

Dosen Pembimbing I :
Prof. Dr. Ir. Hidayat Soegihardjo, MS.

Dosen Pembimbing II :
Data Iranata, S.T., M.T., Ph.D.

DEPARTEMEN TEKNIK SIPIL
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2020

FINAL PROJECT - RC18-4803

DESIGN MODIFICATION OF CINCIN LAMA WIDANG BRIDGE USING *EXTRADOSED* SYSTEM

MUHAMMAD ANHAR PRAKOSO
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Sepuluh Nopember Institute of Technology
Surabaya
2020

LEMBAR PENGESAHAN

MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM *EXTRADOSED*

TUGAS AKHIR

Diajukan Untuk Memenuhi Salah Satu Syarat
Memperoleh Gelar Sarjana Teknik
Pada
Program Studi S-1 Departemen Teknik Sipil
Fakultas Teknik Sipil, Perencanaan dan Kebumian
Institut Teknologi Sepuluh Nopember

Oleh :
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SURABAYA
AGUSTUS, 2020

MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED

Nama Mahasiswa : Muhammad Anhar Prakoso
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Abstrak

Jembatan merupakan sarana transportasi yang dapat memudahkan akses antar lokasi dan menunjang pemerataan pembangunan infrastruktur. Jembatan menghubungkan dua bagian jalan yang terputus karena adanya rintangan – rintangan, seperti lembah yang dalam, alur sungai, danau, saluran irigasi, kali, jalan kereta api, maupun jalan raya yang melintang tidak sebidang. Jembatan Cincin lama merupakan jembatan penghubung antara kecamatan babat lamongan, dan kecamatan widang. Struktur utama jembatan merupakan rangka batang yang memiliki profil WF dengan bentang 260 m. Jembatan Cincin Lama dibagi menjadi lima segmen dengan empat pilar (55 m + 55 m + 55 m + 55 m + 40 m), serta lebar lantai kendaraannya 9 m. Sistem Jembatan rangka batang umumnya diperuntuhkan untuk jembatan dengan bentang 60 -100 m . Hal ini tidak sesuai pada jembatan cincin lama yang menggunakan struktur rangka batang dengan bentang 260 m. Oleh karena itu dilakukan modifikasi pada Jembatan Cincin Lama menggunakan sistem exstradosed.

Jembatan extradose merupakan modifikasi jembatan bentang panjang yang didapatkan dari kombinasi antara jembatan girder dan jembatan cable stay. Jembatan extradose memiliki tinggi tower yang lebih pendek serta sudut kabel antara pylon dan *gelagar relatif lebih landai dibandingkan dengan jembatan cable stay pada umumnya.*

Dalam perencanaan modifikasi ini dimulai dengan melakukan pengumpulan data dan studi literature, kemudian dilanjutkan dengan preliminary desain berupa penentuan dimensi kabel, gelagar, dan pylon. Setelah itu dilanjutkan dengan desain struktur sekunder, permodelan dan analisa struktur, desain struktur utama serta dilanjutkan dengan control stabilitas. Apabila sudah sesuai dengan persyaratan maka perencanaan dilanjutkan dengan melakukan desain perletakan dan expantion joint. Perencanaan modifikasi ini mengacu pada SNI 1725-2016, SNI 2833-2016, dan Peraturan Kementrian PU mengenai Pedoman Perencanaan Teknis Jembatan Beruji Kabel 2015.

Dari hasil perencanaan digunakan lantai kendaraan berupa tapered box gider dengan lebar 16 m dan ketinggian 3 m hingga 4,5 m, dengan tinggi tower setinggi 14 m dari lantai kendaraan. Kabel jembatan menggunakan VSL dengan unit 6-127-115, 6-109-107, 6-85-84, 6-55-52, 6-19-16, 6-61-59, 6-73-64, 6-85-81, 6-109-103, 6-127-114.

Kata kunci : Jembatan, Kabel, Gelagar, Extradosed

DESIGN MODIFICATION OF CINCIN LAMA WIDANG BRIDGE USING *EXTRADOSED* SYSTEM

Student Name : Muhammad Anhar Prakoso
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Abstract

A bridges is a means of transportation for the purpose of providing passage between locations and supporting equitable infrastructure development. Bridge connects two sections of the road that are cut off due to any obstacles, such as deep valleys, river courses, lakes, irrigation channels, rivers, railroad tracks, and crossing uneven roads. Cincin Lama Bridge is a connecting bridge between the Babat district of Lamongan, and the Widang district. The main structure of the bridge is a truss that has a WF profile with 260 m span. Cincin Lama Bridge is divided into five segments with four pillars (55 m + 55 m + 55 m + 55 m + 40 m), and the deck width is 9 m. The truss bridge system is generally intended for bridges with a 60 - 100 m span. This is not suitable for Cincin Lama Bridges which use a trunk structure with 260 m span. Therefore, modifications were made to the Cincin Lama Bridge using Exstradosed System.

Extradose bridge is a modification of a long span bridge which is a combination of the girder bridge and the cable stay bridge. The extradose bridge has a shorter tower height and the cable angle between the pylon and the girder is relatively sloping compared to the cable stay bridge in general.

In the planning of this modification, it starts with collecting data and studying literature, then proceeding it to preliminary design in the form of determining the dimensions of cables, girder, and pylon. After the proceeding with the secondary structure design, it continues with modeling, analysing the structure, *designing the main structure and calculating the stability control. If it is in accordance with the requirements, the planning is continued to design the placement and expansion joint. This modification planning refers to SNI 1725-2016, SNI 2833-2016, and Regulation of the Ministry of Public Works regarding the 2015 Cable Test Bridge Technical Planning Guidelines.*

As a result of the planning, A deck in the form of tapered box gider is used. It has a width of 16 m and a height of 3 m to 4.5 m, with a tower height as high as 14 m from the vehicle floor. Bridge cables use VSL with units 6-127-115, 6-109-107, 6-85-84, 6-55-52, 6-19-16, 6-61-59, 6-73-64, 6- 85-81, 6-109-103, 6-127-114.

Kata kunci : Bridge, Stay, Girder, Extradosed

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Penulis menyadari sepenuhnya bahwa tugas akhir ini tidak akan selesai tanpa bantuan dan dorongan dari berbagai pihak. Maka dari itu, pada kesempatan ini kami menyampaikan terima kasih kepada:

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Surabaya, 05 Juni 2020

Penulis

DAFTAR ISI

“Halaman ini sengaja dikosongkan”

DAFTAR GAMBAR

"Halaman ini sengaja dikosongkan"

DAFTAR TABEL

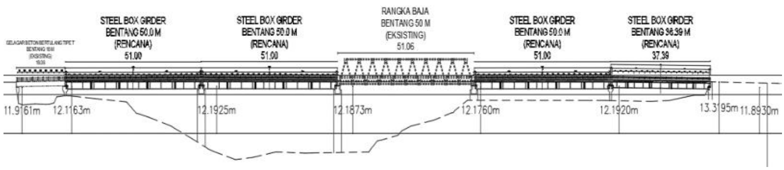
“Halaman ini sengaja dikosongkan”

BAB I
PENDAHULUAN

1.1 Latar Belakang

Pemerataan pembangunan infrastruktur merupakan salah satu indikator perkembangan ekonomi dan majunya suatu daerah, hal ini tidak lepas dari kemudahan akses antar lokasi di wilayah tersebut. Indonesia sebagai negara kepulauan yang memiliki luas perairan mencapai 2/3 dari seluruh luas wilayahnya, yaitu ± 5.877.879 km^2, serta daratan Indonesia dengan luas ±2.012.402 km^2 yang memiliki kondisi ketinggian yang tidak rata, terbukti dengan banyaknya lembah, serta jurang yang menyebabkan terputusnya akses antar daratan (Janhidros,2006 dalam (Ramdhan & Arifin, 2013)) . Oleh karena itu, dibutuhkan adanya sarana transportasi yang mampu menghubungkan antar wilayah tersebut. Jembatan merupakan salah satu alternatif infrastruktur yang dapat memudahkan akses antar lokasi. Jembatan merupakan konstruksi yang menghubungkan dua bagian jalan yang terputus karena adanya rintangan – rintangan, seperti lembah yang dalam, alur sungai, danau, saluran irigasi, kali, jalan kereta api, maupun jalan raya yang melintang tidak sebidang (Jemmy,dkk. 2014).

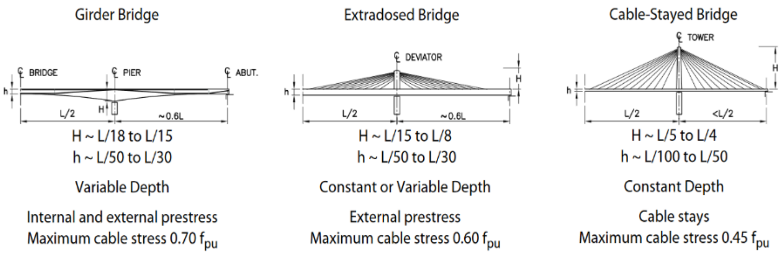
Jembatan Cincin lama merupakan jembatan penghubung antara kecamatan babat lamongan, dan kecamatan widang, tuban. Jembatan ini dibangun pada tahun 1978 dengan bentang bersih 260 m. jembatan cincin lama memiliki struktur utama berupa rangka batang dengan profil WF. Jembatan ini dibagi menjadi lima bagian dengan empat pilar (55 m + 55 m + 55 m + 55 m + 40 m), serta lebar lantai kendaraannya 9 m. Berikut adalah detail perencanaan eksisting pada Jembatan Cincin Lama (Gambar 1.1).



Gambar 1.1 Jenis Rangka Seismik yang Umum digunakan
Sumber : BBPJN VII

Sistem Jembatan rangka batang umumnya diperuntuhkan untuk bentang 60-100m (Binamarga,1997). Hal ini tidak sesuai pada jembatan cincin lama yang memiliki bentang 260 m dan menggunakan struktur rangka batang. Oleh karena itu, pada jembatan cincin lama terdapat modifikasi dengan membagi bentang jembatan menjadi beberapa segmen. Hal ini menyebabkan penggunaan banyak pilar pada struktur bawah dari jembatan cincin lama. Penggunaan banyak pilar mengakibatkan sulitnya metode pelaksanaan dalam konstruksi jembatan dilapangan. Maka dibutuhkan adanya modifikasi jenis jembatan yang mampu meminimalisir penggunaan pilar.

Jembatan extradose merupakan modifikasi jembatan bentang panjang yang didapatkan dari kombinasi antara jembatan girder dan jembatan cable stay. Jembatan extradose memiliki tinggi tower yang lebih pendek serta sudut kabel antara *pylon* dan gelagar relatif lebih landai dibandingkan dengan jembatan *cable stay* pada umumnya (Benjumea,dkk.,2010). Berikut adalah spesifikasi perbandingan antara jembatan girder, jembatan *cable stay*, serta jembatan *extradosed* (Gambar 1.2).



Gambar 1.2 Tipe Jembatan Extradosed
Sumber : Mermigas.,2008

Metode pada Jembatan Extradose dinilai lebih effektif dan efisien bila dibandingkan dengan jembatan girder maupun jembatan *cable stay*. Hal ini dikarenakan pada jembatan tipe ini tinggi *pylon* yang digunakan pada jembatan lebih pendek di bandingkan dengan cable stay dan sudut kabel antara *pylon* dan gelagar relatif landai dibandingkan dengan *cable stay* sehingga memudahkan dalam pelaksanaan. Selain itu, nilai *allowable stress rat*io untuk jembatan *extradosed* diijinkan sampai 60%, angka ini lebih besar dari pada jembatan *cable-stayed* yang hanya memiliki *allowable stress rat*io sebesar 45% (Benjumea, 2010).

Sebagai bahan studi perencanaan, maka akan dilakukan modifikasi pada Jembatan Cincin Lama Lamongan-Tuban Jawa Timur Indonesia. Jembatan yang sebelumnya menggunakan struktur utama rangka batang menjadi jembatan *extradosed*. Modifikasi perencanaan ini dilakukan agar didapatkan desain jembatan yang mampu meminimalkan kebutuhan pilar, sehingga memudahkan pelaksanaan konstruksi di lapangan. Selain itu, jembatan tipe extradose memiliki nilai estetika yang lebih tinggi bila dibandingkan dengan jembatan rangka batang, hal ini didapatkan dari komposisi desain lantai kendaraan, susunan kabel, dan perencanaan pilar jembatan.

1.2 Rumusan Masalah

Berdasarkan latar belakang di atas dapat dirumuskan permasalahan sebagai berikut:

1. Bagaimana perubahan dari denah dan tampak jembatan setelah modifikasi ?
2. Bagaimana merencanakan *preliminary design* jembatan *extradosed* ?
3. Bagaimana pembebanan pada desain jembatan setelah modifikasi ?
4. Bagaimana merencanakan lantai kendaraan, kabel, dan tiang dengan desain *extradosed* ?
5. Bagaimana permodelan dan analisa struktur dalam perencanaan jembatan *extradosed* ?
6. Bagaimana mengontrol jembatan ini akibat dari beban yang ada ?
7. Bagaimana menuangkan hasil desain struktur Jembatan Cincin Lama ?

1.3 Tujuan

Dengan rumusan masalah tersebut maka tujuan yang diharapkan adalah sebagai berikut:

1. Merencanakan perubahan denah dan tampak jembatan setelah modifikasi.
2. Dapat merencanakan *preliminary design* jembatan *extradosed*.
3. Menganalisa pembebanan pada desain jembatan setelah modifikasi.
4. Merencanakan lantai kendaraan, kabel, dan tiang dengan desain *extradosed.*
5. Memodelkan dan menganalisa struktur dalam perencanaan jembatan *extradosed.*
6. Mampu mengontrol jembatan ini akibat dari beban yang ada.
7. Menaungkan hasil desain struktur Jembatan Cincin Lama dalam bentuk gambar teknik.

1.4 Batasan Masalah

Mengingat keterbatasan waktu dan keterbatasan data dalam penyusunan tugas akhir ini maka perencanaan Jembatan Cincin Lama diperlukan pembatasan masalah yang meliputi:

1. Tidak merencanakan struktur bawah.
2. Tidak merencanakan perkerasan dan geometric jalan.
3. Tidak memperhitungkan analisis hidrologi.
4. Tidak meninjau aspek *mechanical* dan *electrical.*
5. Tidak memperhitungkan anggaran biaya.

1.5 Manfaat

Manfaat yang dapat diperoleh dari penyusuna tugas akhir ini adalah:

1. Desain baru Jembatan Cincin Lama diharapkan dapat menjadi referensi dalam melakukan desain jembatan dengan menggunakan system *extradosed.*
2. Untuk dunia ketekniksipilan, diharapkan desain baru Jembatan Cincin Lama menjadi inspirasi jembatan masa

depan yang lebih modern untuk diterapkan khususnya di Indonesia.

3. Untuk penulis diharapkan dapat menambah pengetahuan dan *skill* ilmu perencanaan jembatan.

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BAB II
TINJAUAN PUSTAKA

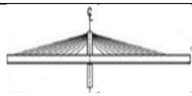
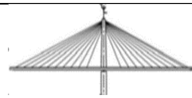
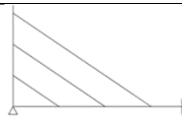
2.1 Jembatan Extradosed

Jembatan Extradosed merupakan salah satu jenis jembatan bentang panjang yang memiliki kemiripan dengan jembatan cable stay di mana kabel dari pylon yang terhubung pada gelagar digunakan sebagai penguatan. Jembatan Extradosed sendiri merupakan sebuah konsep yang diperkenalkan oleh J. Mathivat pada tahun 1988. Jembatan Extradosed merupakan peralihan antara jembatan dengan tipe gelagar dan cable stay. Ada pula karakteristik dari jembatan extradosed sebagai berikut (Jingxian dkk, 2018).

1. Dari segi estetika : tinggi dari balok jembatan extradosed memiliki ketinggian ½ dari jembatan gelagar biasa. Dan tinggi tower extradosed lebih pendek dibandingkan dengan cable stay. Sehingga memilik kesan estetika yang lebih indah dari pada jembatan gelagar biasa.
2. Pengaturan bentang yang fleksibel : jembatan extradosed dapat dirancang sebagai menara tunggal dengan bentang ganda, dua menara dengan tiga bentang, dan multi menara dengan multi bentang. Jarak bentang tunggal sendiri berkisar 100 m –300 m.
3. Konstruksi yang sederhana : Jembatan extradosed dapat dibangun dengan metode pengecoran kantilever, yang pada dasarnya sama dengan jembatan gelagar biasa. Tidak perlu menyesuaikan kekuatan kabel sekunder selama konstruksi. Karena menara jembatan extradoesd relative pendek, dan pembangunan menara jembatan tidak serumit jembatan cable stay.
4. Dari segi ekonomi : berdasarkan analisa biaya pembangunan jembatan extradosed di seluruh dunia. Biaya jembatan per meter dari jembatan extradosed pada dasarnya sama dengan pembuatan jembatan girder dan lebih rendah dibandingkan

jembatan cable stay pada umunya. Akibatnya, ia memiliki manfaat ekonomi yang cukup besar.

Tabel 2.1 Perbandingan Tiga Tipe Jembatan

	Jembatan Girder	Jembatan Extradosed	Jembatan Cable-Stayed
Tipe Layout			
Pengaturan Kabel			
Diagram Geser			
Ketebalan Girder	L/50 – L/15	L/50 – L/30	L/100 – L/50
Ketinggian Tower	NA	L/15 – L/8	L/5 – L/4
Prestress	Internal dan eksternal prestress	Eksternal prestress	Cable stays
Max cable stress	NA	0,60 f_{pu}	0,45 f_{pu}

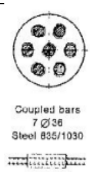
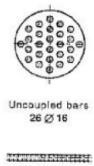
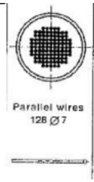
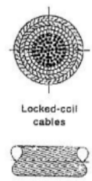
Sumber : Jiong Hu dkk, 2016

2.2 Kabel Jembatan Extradosed

Konfigurasi kabel merupakan bagian yang penting dalam desain jembatan dengan system kabel karena hal ini akan mempengaruhi tidak hanya pada kinerja structural jembatan tetapi juga menyangkut metoda/kemudahan ereksi dan biaya pembangunan. (Mustazir, 2002).

Secara umum bahan baja yang digunakan untuk kabel jembatan ditandai dengan kandungan karbon yang lebih tinggi dibandingkan dengan baja structural. Bila kandungan karbon dari baja structural adalah 0,15% – 0,20%, kandungan karbon dari bahan yang digunakan pada kabel jembatan extradosed sama dengan 0,80% (Gimsing dan Georgakis 2011). Karena kandungan karbonnya yang lebih tinggi, kekuatan Tarik material kabel berkisar 1800 MPa, hamper sama dengan empat kali lipat dari baja structural (370 MPa). Dengan tingginya kuat tarik pada kabel jembatan extradosed juga memiliki efek samping yaiut menurunnya daktalitas. Perpanjangan pada titik putus untuk bahan kabel sama dengan 4%. Sedangkan untuk baja structural, perpanjangan pada titik putus bias mencapai 24% (Gimsing dan Georgakis 2011). Untuk macam-macam tipe kabel dapat dilihat pada **Gambar 2.1 – Gambar 2.4** dan spesifikasi pada **Tabel 2.2**.

Tabel 2.2 Spesifikasi Kabel

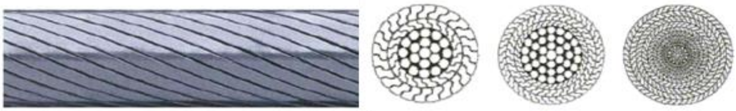
Jenis Kabel	 Coupled bars 7 Ø 36 Steel 835/1030	 Uncoupled bars 26 Ø 16	 Parallel wires 128 Ø 7	 Strands 27 Ø 15, 15 mm	 Locked-coil cables
Tendons	*Bars* ϕ26.5, 32, 36 mm	*Bars* ϕ 16 mm	*Wire* ϕ 6, 7 mm	*Strand* ϕ 0.5, 0.6, 0.7 of 7 *twisted wires*	*Wire with different profiles* ϕ2 9-7 mm
0.2% proof stress, σ_{02} (N/mm^2)	835 1080	1350	1470	1570 ~ 1670	-
Ultimate tensile strength, β_z (N/mm^2)	1030 1230	1500	1670	1770 ~ 1870	1000 ~ 1300
Fatigue					
$\Delta\sigma$ (N/mm^2)	80	-	350	300 ~ 320	120 ~ 150
$\sigma max/\beta_z$	0.6	-	0.45	0.5 ~ 0.45	0.45
Modulus of elasticity, E (N/mm^2)	210 000	210 000	205 000	190 000 ~ 200 000	180 000 ~ 165 000
Failure Load kN	7339	7624	7467	7634	7310

Sumber: Peraturan PUPR 08/SE/M/2015

a. *Spiral Strand*
Spiral Strand terdiri dari beberapa kawat yang dipuntir bersama-sama. Kawat-kawat tersebut diuntai dalam satu atau lebih layer, biasanya dengan arah yang berlawanan, untuk memperoleh diameter yang dibutuhkan.

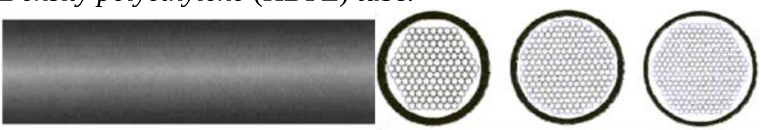
Gambar 2.1 Kabel Tipe Spiral Strand
Sumber : Mustazir, 2002

b. *Locked Coil Strand*
Locked Coil Strand memiliki inti yang terbuat dari *Spiral Strand* yang kemudian diuntai oleh kawat-kawat persegi hingga mencapai diameter yang dibutuhkan



Gambar 2.2 Kabel Tipe Locked Coil Strand
Sumber : Mustazir, 2002

c. *Parallel Wire Strand*
Parallel Wire Strand terdiri dari kawat dengan diameter antara 5 mm – 7 mm yang memiliki bentuk hexagonal, dengan suatu helix panjang. Seluruh kawat tersebut dibungkus oleh *High Density polyethylene* (HDPE) tube.



Gambar 2.3 Kabel Tipe Paralel Wire Strand
Sumber : Mustazir, 2002

d. *Structural Rope*
Structural Rope biasanya terdiri dari 6 buah *strand* (untaian kawat) ayng dipuntir mengelilingi *steel core*. Kawat ini memiliki diameter yang relative kecil sehingga memiliki

kelenturan yang tinggi. Biasanya kabel tipe ini memiliki modulus elastisitas kurang dari setengah modulus elastisitas baja structural.

Gambar 2.4 Kabel Tipe Struktural Rope
Sumber : Mustazir, 2002

2.2.1 Susunan Tipe Kabel

Pada sistem jembatan *Extradosed* memiliki beberapa variasi penempatan susunan kabel. Susuna kabel pada jembatan *Extradosed* tidak hanya berefek pada kekuatan struktur tetapi juga terhadap kemudahan metode pelaksanaan serta biaya. Berikut dalah klasifikasi susunan kabel dari jembatan *Extradosed* (Troitsky, 1998) :

1) *Radial or Converging System*
   Pada sistem ini semua kabel diarahkan ke puncak menara. Secara structural, pengaturan ini memiliki keunggulan, karena memiliki kemiringan maksimum terhadap horizontal. Akibatnya kabel menerima langsung beban mati dan hidup secara langsung. Tetapi, ketika sejumlah kabel berada bersamaan di puncak menara, kabel pendukung atau sadel di dalam menara mungkin sangat padat. Sehingga perincian menjadi lebih rumit.
2) *Harp or Parallel System*
   Pada sistem ini kabel dihubungkan ke menara pada ketinggian yang berbeda, dan ditempatkan secara sejajar satu sama lain. Secara estetika sistem tipe ini lebih disukai. Kabel berbentuk harpa ini dapat memberikan kekakuan yang sangat baik pada bentang utama jembatan. Jumlah baja yang dibutuhkan untukpengaturan kabel berbentuk harpa sedikit lebih tinggi dari pada yang berbentuk kipas. Sehingga kurva kuantitas baja

menyarankan untuk memilih menara yang lebih tinggi yang juga meningkatkan kekakuan sistem kabel terhadap defleksi.

3) *Fan or Intermediate System*
Susunan kabeldengan tipe *fan* atau *intermediate* merupakan modifikasi dari sistem harpa. Gaya pada kabel tetap kecil sehingga tali tunggal dapat digunakan. Semua kabel memiliki *fixed connection* terhadap menara.

4) *Star System*
Star pattern merupakan susunan kabel yang menarik secara estetika. Namun, hal tersebut bertentangan dengan prinsip bahwa titik-titik pemasangan kabel harus didistribusikan sebanyak mungkin di sepanjang girder utama.

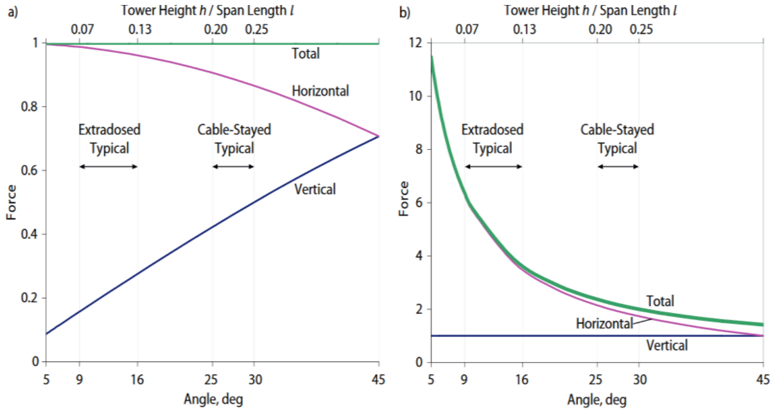
Tabel 2.3 Pengaturan Sistem Kabel

STAY SYSTEM		SINGLE 1	DOUBLE 2	TRIPLE 3	MULTIPLE 4	VARIABLE 5
1	BUNDLE OR CONVERGING OR RADIAL					
2	HARP OR PARALLEL					
3	FAN					
4	STAR					

Sumber : Troitsky, 1998

2.2.2 Konfigurasi Kabel

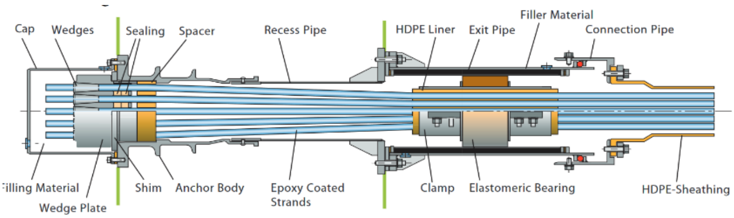
Dalam sebuah jembatan *extradosed* komponen vertical didukung dari gaya pada kabel yang mengangkat girder menerus, sementara komponen horizontal didukung *prestresses* gelagar. Konfigurasi kabel dan ketinggian menara adalah dua factor yang mempengaruhi kemiringan kabel, dan perilaku kabel di dek. Dua hal ini merupakan factor yang akan dibahas dalam kaitannya dengan jembatan *extradosed*. Jarak antar kabel untuk beton 5-10m dan untuk baja 15-25m (pedoman jembatan kabel 2015). Berikut adalah grafik pengaruh sudut kabel pada jembatan *extradosed* yaitu pada Gambar 2.5.



Gambar 2.5 Pengaruh kemiringan kabel terhadap gaya
Sumber : Mermigas, 2008

2.2.3 *Anchorage*

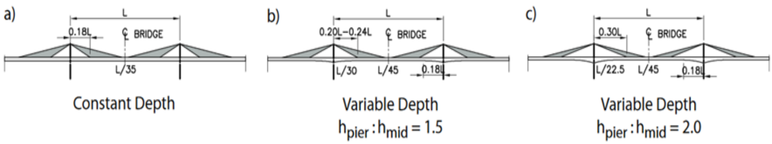
Pada jembatan *extradosed* rentang tegangan untuk beban hidup dibatasi pada 80 MPa, untuk angkur pratekan konvensional dapat digunaan sebagai angkur jembatan dari pada angkur *cable-stayed* yang biasanya dirancang untuk tegangan 200 MPa hingga 250 MPa. Dywidag-Systems International memasarkan *Extradoesd Anchorage*, jenis XD (-E untuk epoxy dilapisi helai), yang ditunjukan pada gambar 2.6, yang sengaja dirancang untuk menggabungkan kepala angkur pada tendon eksternal dengan rincian perlindungan yang ditemukan di DYNA-Grip dan DYNA-Bond Stay Cables (Dywidag 2006). Untuk tiga kabel tersebut, bantalan elastomer yang terkandung dalam pipa reses untuk mencegah tekana lentur pada kepala angkur. Berikut adalah gambar angkur jenis XD-E.



Gambar 2.6 Angkur kabel tipe XD-E
Sumber : Dywidag 2006

2.3 Panjang Side Span

Chio Cho (2000) menemukan bahwa bentang sisi kurang dari setengah bentang utama dapat mengurangi momen lentur dalam bentang utama, tetapi direkomendasikan penggunaannya 0,60 dari bentang utama agar menghasilkan momen lentur positif dalam bentang sisi karena beban hidup yang relatif sama dengan bentang utama. Berikut adalah cara menentukan *main span* dan *side span* pada Gambar 2.6.

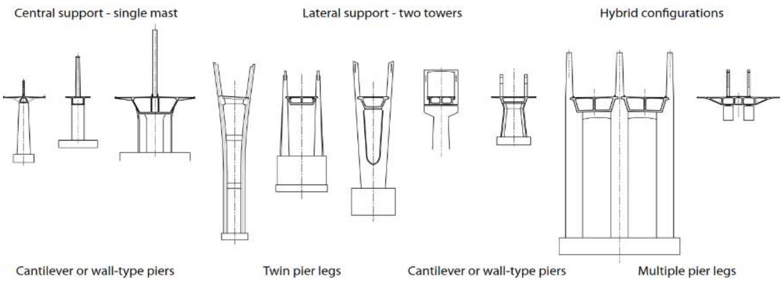


Gambar 2.7 Geometri Jembatan Extradosed
Sumber : Mermigas, 2008

2.4 Tower dan Piers

Desain menara memungkinkan untuk ekspresi dan kreativitas pada structural jembatan. Keputusan penting dalam desain menara adalah apakah menggunakan tiang tunggal atau dua penyangga lateral. Keputusan ini harus dibuat bersama dengan pemilihan penampang, dan pengaturan kabel. Konfigurasi kabel harpa menyebabkan tekukan signifikan pada menara mengharuskan lebar minimum (Mermigas, 2008).

Sebuah tiang tunggal diatas dek akan lebih ekonomis dari pada dua tiang. Tiang tunggal memungkinkan untuk *pier* tunggal dengan lebar yang relatif sempit dan didukung oleh pondasi tunggal, dibandingkan dengan dua tiang yang biasanya diperpanjang hingga ke permukaan tanah dan diangkurkan dalam satu pondasi besar (Mermigas, 2008). Beberapa desainer telah memilih untuk mentransisikan tiang lateral ke dalam kolom *pier* tunggal dengan menggunkan balok transversal yang dalam dan menghasilkan konfigurasi yang menyerupai garpu tala. Gambar 2.8 menunjukan berbagai bentuk *pier* dan menara yang telah digunakan dalam jembatan *extradosed*. Berikut adalah gambar tipe-tipe *pylon* atau *pier* pada jembatan *extradosed*.



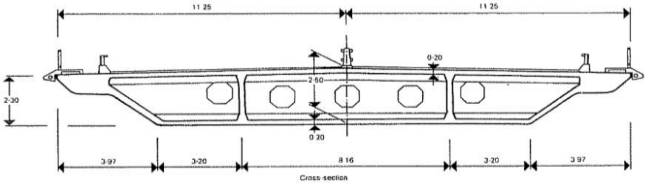
Gambar 2.8 Tipe pylon atau pier pada jembatan extradosed
Sumber : Mermigas, 2008

2.5 Lantai Kendaraan

Penggunaan sistem lantai kendaraan dapat menambahkan kekakuan sari konstruksi jembatan *extradosed*. Lantai kendaraan dapat berupa beton, *orthotropic*, atau baja yang sebagian diisi dengan beton (komposit baja-beton). Pada sistem lantai juga terdapat pengaruh kembang susut yang dapat menyebabkan penambahan tegangan pada struktur dek itu sendiri. Unutk itu penggunaan *expantion joint* diberikan setiap 30-40 meter untuk mencegah kerusakan dek dan struktur utama (Triotsky, 1994).

2.5.1 Lantai Kendaraan dari Beton

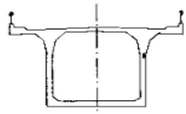
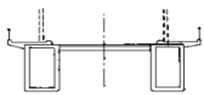
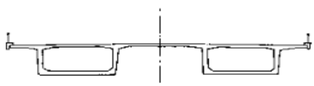
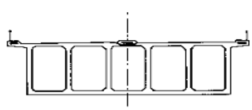
Lantai jembatan jenis ini dapat dibuat secara *precast* maupun *cast in place* dikarenakan berat sendiri yang cukup besar, lantai jenis ini cocok untuk bentang menengah dan panjang. Lantai ini dapat berupa beton bertulang maupun beton prategang. Biaya yang dikeluarkan untuk pembuatan lantai jembatan jenis ini tergolong murah, namum berat sendiri pada lantai jembatan mengakibatkan adanya tambahan beban mati pada jembatan. Hal ini menjadikan perlu untuk memperbesar dimensi dari kabel, tiang, dan pengangkeran pada sturktur jembatan (Juvani, 2012). Untuk contoh lantai kendaraan dari beton dapat dilihabt pada Gambar 2.9.



Gambar 2.9 Penampang gelagar utama beton
Sumber : Walther, 1999

Bentuk-bentuk gelagar utama dengan material beton dapat dilihat pada Tabel 2.4

Tabel 2.4 Penampang gelagar beton

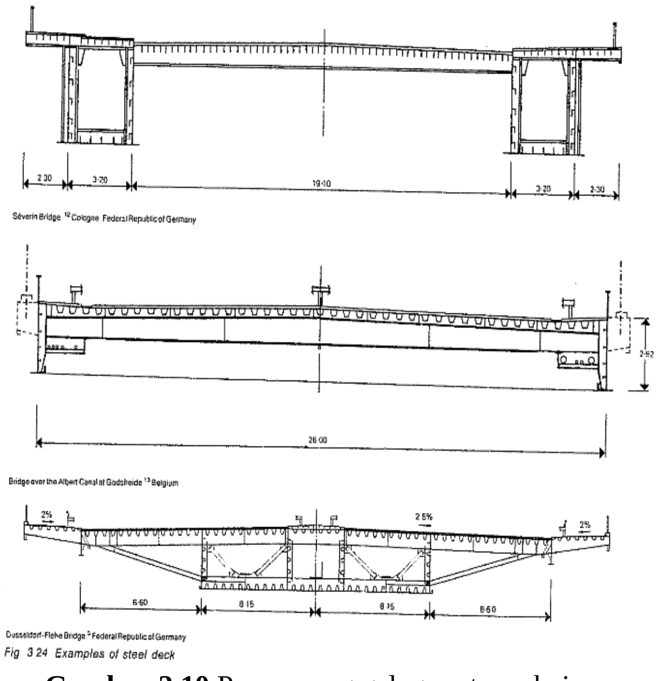
	Type of girder	Deck cross - section
1	Single box girder (Wadi Kuf Bridge, Libya)	
2	Twin box girders (River Parana Bridge, Argentina)	
3	Twin box girders (River Waal Bridge, Holland)	
4	Multiple box girder (Polcevera Viaduct, Italy)	

Sumber : Troitsky, 1988

2.5.2 Lantai Kendaraan dari Baja

Penggunaaan baja sebagai material gelagar utama bisa mengurangi keseluruhan berat struktur jika dibandingkan dengan menggunakan beton karena berat sendiri sturktur baja lebih kecil daripada beton. Hal ini memungkinkan untuk mendesain jembatan lebih lebar untuk digunakan pada jembatan *cable stayed* bentang panjang (Juvani, 2012).

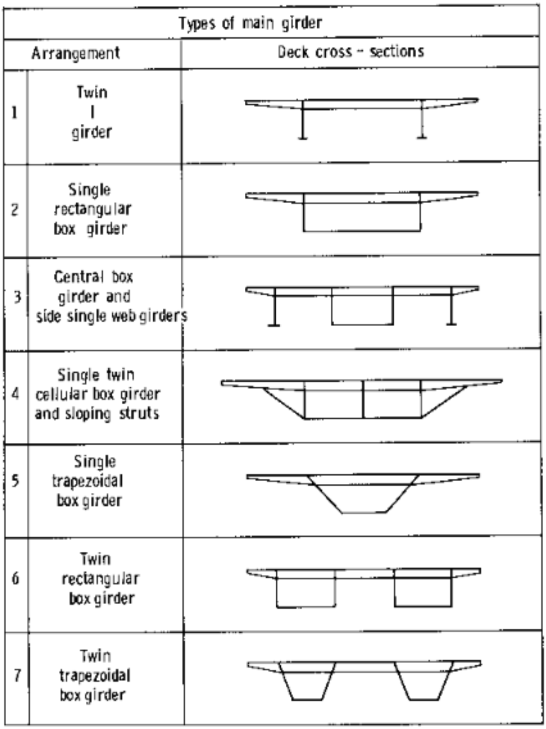
Untuk contoh lantai kendaraan dari baja dapat dilihat pada Gambar 2.10.



Gambar 2.10 Penampang gelagar utama baja
Sumber : Walther, 1999

Bentuk-bentuk gelagar utama dengan material baja dapat dilihat pada Tabel 2.5.

Tabel 2.5 Penampang gelagar baja



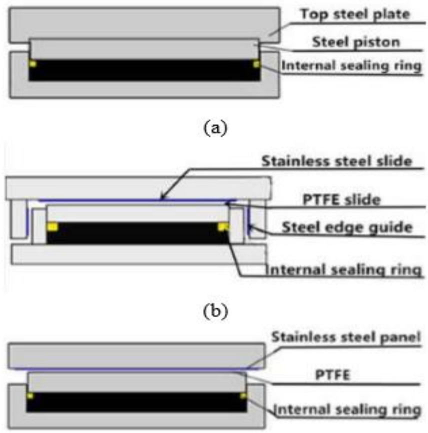
Types of main girder		
Arrangement		Deck cross – sections
1	Twin I girder	
2	Single rectangular box girder	
3	Central box girder and side single web girders	
4	Single twin cellular box girder and sloping struts	
5	Single trapezoidal box girder	
6	Twin rectangular box girder	
7	Twin trapezoidal box girder	

Sumber : Troitsky,1988

2.6 Perletakan Jembatan

Pot bearing digunakan sebagai perletakan untuk konstruksi yang memiliki beban tinggi, pergeseran (Deflection) yang besar dan rotasi yang tinggi. Hal ini dikarena pot bearing dapat mengatasi beban vertikal yang cukup besar sementara yang membutuhkan sedikit ruang, terutama dalam hal ketebalan. Pot bearing pada dasarnya terdiri dari elastomer tertahan dari pot logam. Pad ini kemudian ditekan oleh piston dengan bantalan yang menahan geser atau rotasi, tergantung pada desain yang dibutuhkan. Kemampuan gerakan dari pot bearing dapat diberikan dalam satu atau dua arah.

Pot bearing dibagi menjadi tiga jenis, yaitu: Fixed (TF), Guided (TGe), dan Free Sliding (TGa). Untuk lebih jelasnya bisa dilihat pada Gambar 2.11.



Gambar 2.11 Tipe-tipe pot bearing, (a) Fixed, (b) Guided, (c) Free Sliding

Sumber : Dacheng Rubber Pot Bearing Brochure

1. *Fixed Pot Bearing*

   Merupakan jenis pot bearing yang tidak dapat bebas bergerak ke segala arah. Saat dibebani, pot bearing ini berperilaku seperti cairan terjepit yang tidak dapat ditekan sehingga menjadi penghalang untuk berputar ke sumbu manapun.

2. *Guided Pot Bearing*

   Dengan adanya guide edge hanya bisa bergerak searah. Stainless Steel Slide dan PTFE Slide mengurangi koefisien gesekan dan pot bearing ini biasa digunakan ketika gaya horizontal sebuah struktur relatif kecil (kurang dari 20% dari gaya vertikalnya).

3. *Free Sliding Pot Bearing*

   Dapat bergerak bebas ke segala arah. Penambahan PTFE dan Stainless Steel sliding diantara piston dan pelat dasar, menciptakan sebuah bantalan yang mengizinkan pergerakan horizontal ke segala arah.

Keuntungan menggunakan Pot Bearing, yaitu:

- Daya tahan yang tinggi terhadap gaya horizontal yang besar dan mampu mendistribusikannya dengan aman.
- Daya tahan yang tinggi terhadap beban dinamis dan siklus “fatigue”.
- Mengakomodasi rotasi.
- Tersedia kapastias bervariasi dari 50 ton s/d 10000 ton.

2.7 Expantion Joint

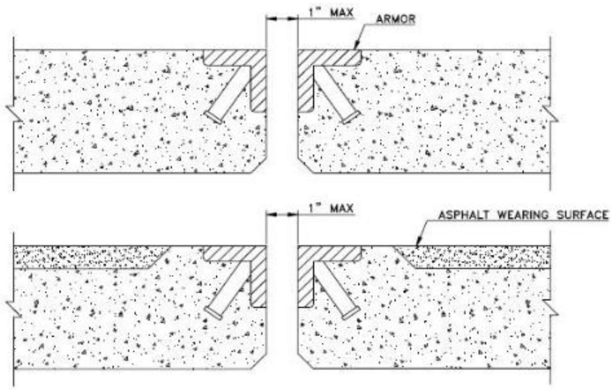
Expansion Joint atau Siar Muai adalah bahan yang dipasang di antara dua bidang lantai beton untuk kendaraan atau pada perkerasan kaku dan dapat juga pertemuan antara konstruksi jalan pendekat sebagai media lalu lintas yang akan melewati jembatan, supaya pengguna lalu lintas merasa aman dan nyaman (Badan Litbang PU, Pd T-13-2005-B). Fungsi dari expansion joint adalah untuk mengakomodasi gerakan yang terjadi pada bagian superstruktur jembatan. Gerakan ini berasal dari beban hidup, perubahan suhu, dan sifat fisik dari pembentuk jembatan). Terdapat 2 jenis expantion joint yaitu joint terbuka dan joint tertutup.

2.7.1 Expantion Joint Terbuka

Pada expansion joint terbuka, sistem drainase diletakan di bawah joint untuk mengumpulkan dan membawa air ke pembuangan. Hal ini dilakukan untuk mencegah kerusakan pada struktur beton. Sistem drainase sendiri berbentuk palung dan dibuat dari bahan anti karat. Jenis expansion joint terbuka yang umum digunakan di Indonesia adalah Butt Joint dan Finger Joint.

a. Butt Joint

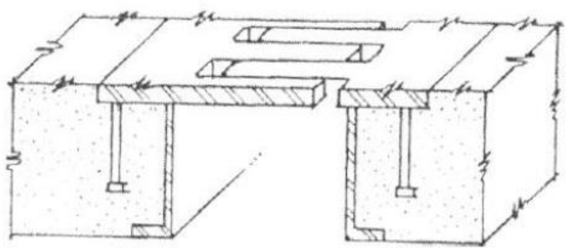
Butt joint adalah *joint* yg menggunakan besi berbentuk siku untuk melindungi tepi beton dari kerusakan akibat kendaraan yang melintas. Joint ini digunakan untuk jembatan dengan small movement, dengan gap maksimum sebesar 25 mm. Butt Joint dibuat dari besi siku yang disebut armor untuk melindungi bagian tepi beton dan dipasangkan pada beton menggunakan stud atau baut. Di Negara barat Butt Joint tidak digunakan lagi karena tidak kedap air. Tapi di Indonesia sendiri masih digunakan untuk jembatan-jembatan pendek. Berikut adalah ilustrasi Expantion joint jenis Butt joint pada Gambar 2.12.



Gambar 2.12 Tipe Expantion joint jenis Butt joint
Sumber : Florida Department of Transportation

b. Finger Joint

Finger Joint bisa mengakomodasi movement mulai dari 75 mm. *Finger Joint* terbuat dari baja dan berbentuk seperti 2 sisir yang saling mengikat. Untuk lebih jelasnya lihat Gambar 2.13 berikut :



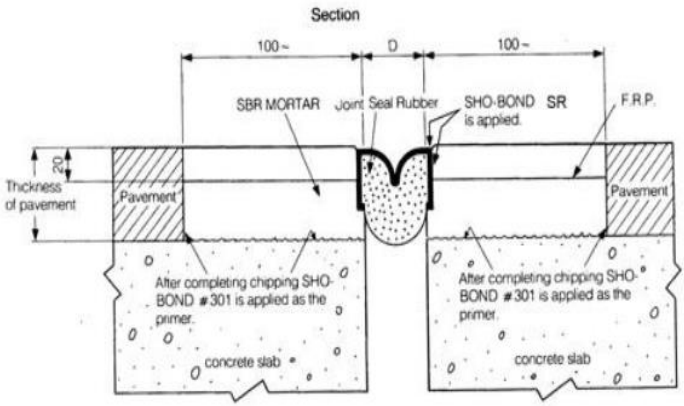
Gambar 2.13 Tipe Expantion joint jenis Finger joint
Sumber : TransportationResearch Board, 2003

2.7.2 Expantion Joint Tertutup

Jenis expansion joint tertutup yang biasa dipakai di Indonesia adalah *New Cut Off Joint, Asphaltic Plug Joint, Strip Seal Joint*, dan *Modular Joint.*

a. New Cut Off Joint

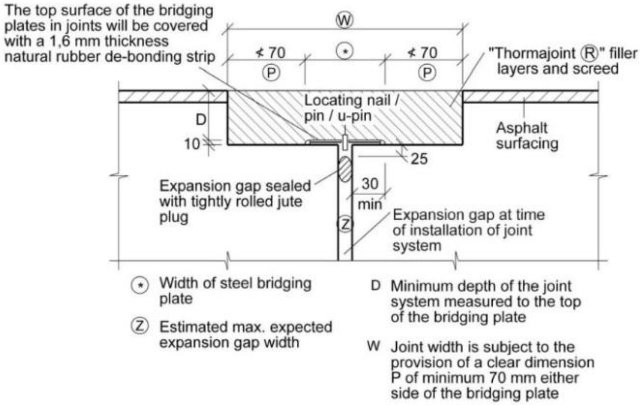
New Cut Off Joint adalah expansion joint yang menggunakan seal berbahan dasar karet. Seal diletakan diantara gap untuk menahan. Untuk lebih jelasnya lihat Gambar 2.14 berikut:



Gambar 2.14 Tipe Expantion joint jenis New Cut Off Joint
Sumber : (Hibondconstruction, 2019)

b. Asphaltic Plug Joint

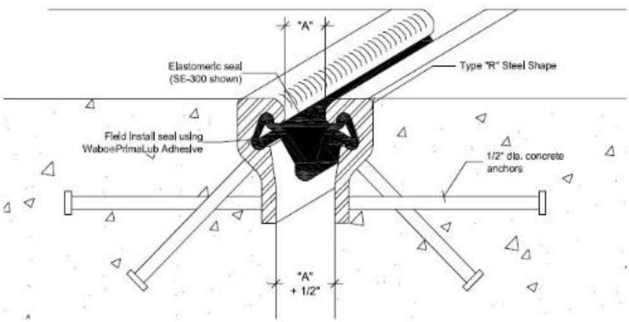
Asphaltic Plug Joint adalah sambungan siar muai yang terbuat dari bahan agregat yang dicampur dengan bahan pengikat binder, pelat baja dan angkur, dibuat pada tempratur tertentu yg berfungsi sebagai bahan pengisi pada sambungan. Untuk lebih jelasnya lihat Gambar 2.15 berikut :



Gambar 2.15 Tipe Expantion joint jenis Asphaltic Plug Joint
Sumber : Agrement, 2003

c. Strip Seal Joint

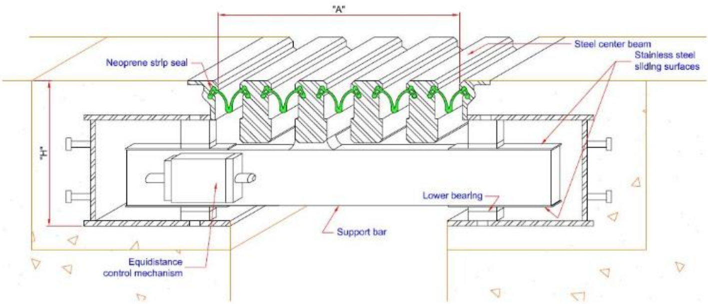
Strip Seal Joint berbentuk strip yag terbuat dari elastomer yang dimasukan ke dalam besi yang ditanam ke pelat beton. *Strip Seal Joint* mempunyai beberapa tipe untuk beragam movement. Ukuran Strip Seal Joint terbesar bisa menangani movement hingga 125 mm, tetapi untuk keamanan kebanyakan orang hanya membatasi hingga 100 mm saja. Untuk lebih jelasnya lihat Gambar 2.16 berikut :



Gambar 2.16 Tipe Expantion joint jenis Strip Seal Joint
Sumber : Watson Bowman Acme, 2000

d. Modular Joint

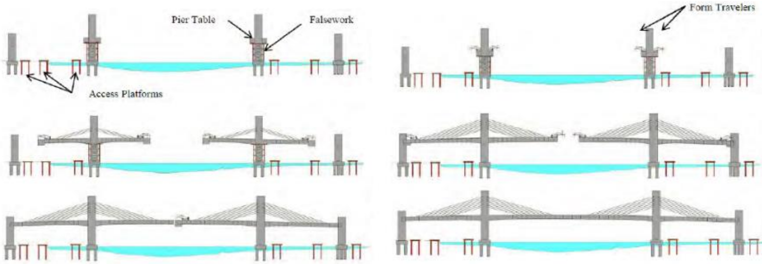
Modular Joint berbentuk sepeti gabungan dari dua atau lebih Strip Seal Joint untuk mengakomodasi movement yang sangat besar. Modular Joint dibuat untuk mengakomodasi movement lebih dari 100 mm. Besarnya modular joint tergantung besarnya movement. Modular joint dirancang untuk jembatan dengan bentang yang panjang dengan kemampuan movement sampai 2 m. Biasanya modular joint digunakan untuk movement antara 150 mm sampai 600 mm. Ada 3 bagian utama dari joint ini, yaitu: sealer, separator beam, dan support bar (Transportation Research Board, 2003). Untuk lebih jelasnya lihat Gambar 2.17 berikut :



Gambar 2.17 Tipe Expantion joint jenis Modular Joint
Sumber : Watson Bowman Acme, 2000

2.8 Metode Pelaksanaan

Sebagian besar jembatan *extradosed* menggunakan metode konstruksi yang sama, yaitu *free balanced cantilever* (Stroh 2012). Metode ini sering digunakan untuk jemabatan dengan bentang menengah atau panjang (200 hingga 500 feet). *Free balanced cantilever* terdiri dari pembagian struktur jembatan menjadi beberapa bagian bentang segmen jembatan. Biasanya menggunakan *cast in-situ* dengan panjang 10 hingga 15 feet per bagian. Bentang membentang dari bagian tower dengan panambahan bagian-bagian segmen hingga bertemu pada tengah bentang. Metode *free balanced cantilever* sangat tepat digunakan ketika tidak memungkinkan untuk mendirikan perancah ditengah bentang seperti Gambar 2.18 (Jiong Hu dkk, 2016).



Gambar 2.18 Metode free balanced cantilever
Sumber : Jiong Hu dkk, 2016

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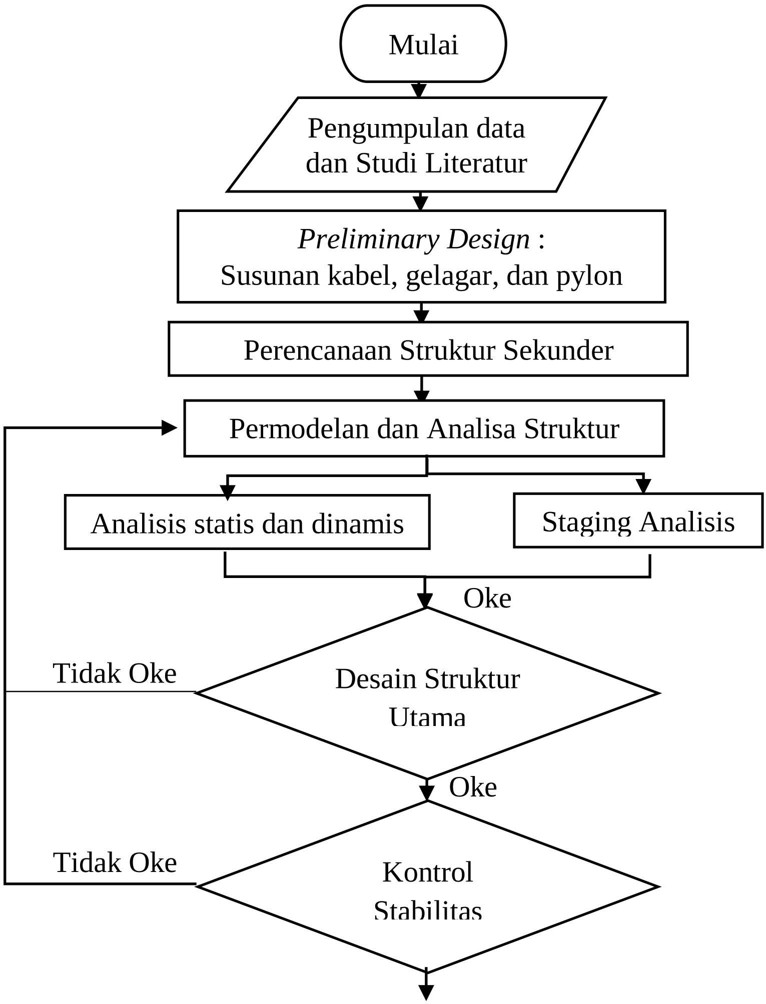
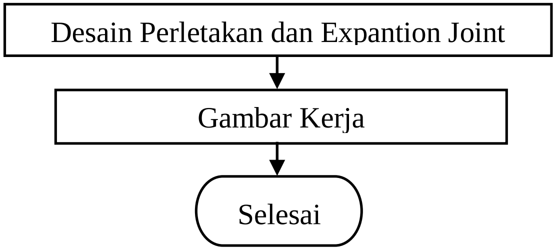
BAB III
METODOLOGI

3.1 Diagram Alir

Prosedur pengerjaan dalam menyelesaikan perencanaan jembatan ini adalah sebagai berikut :





Gambar 3.1 Bagan Metodologi Perencanaan Tugas Akhir

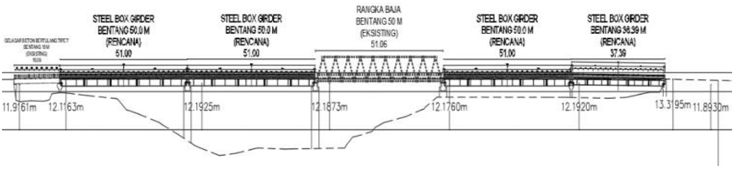
3.2 Pengumpulan Data dan Studi Literatur

Dalam studi literature penulis menggunkana beberapa referensi terkait, berupa jurnal, modul kuliah, buku, peraturan, maupun artikel di internet. Berikut rinciannya :

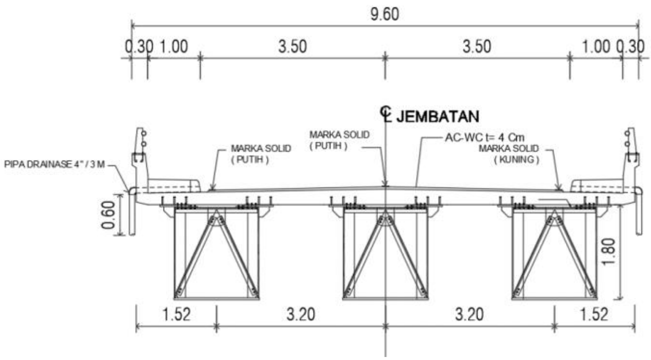
- **Data perencanaan**
  1. Nama dan lokasi : Jembatan Cincin Lama
  2. Bentang pada kondisi *existing* : 37,4 + 51,2 + 51,2 + 51,2 + 51,2 + 18,3 m
  3. Lebar Perkerasan : 7 m
  4. Trotoar : 1+1 m
  5. Muka air banjir : 7,4 m
  6. Jumlah pilar *existing* : 5 buah
  7. Material utama pada kondisi *existing* :
     - Struktur atas berupa rangka batnag
     - Bangunan bawah berupa beton bertulang
- **Rencana Modifikasi :**
  1. Nama dan lokasi : Jembatan Cincin Lama
  2. Bentang pada rencana :
     - 130 m pada main span
     - 65 m pada side span
  3. Lebar pada kondisi rencana : 14 m
  4. Trotoar : 1+1 m
  5. Muka air banjir : 7,4 m
  6. Jumlah pilar rencana : 2 buah
  7. Material utama perencanaan :
     - Struktur atas berupa beton (Box girder)

- Pylon menggunakan sistem H plane
- Sistem kabel semi harp pattern

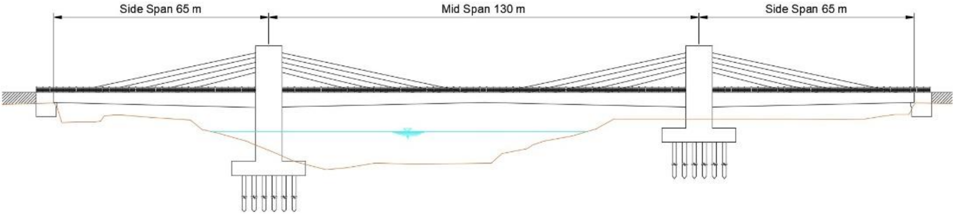
Gambar 3.2 Peta Lokasi Jembatan Cincin Lama Widang
Sumber : Google Maps



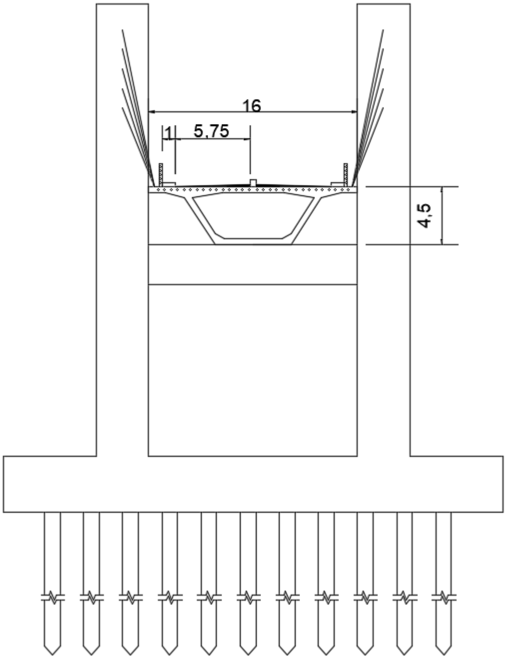
Gambar 3.3 Tampak Memanjang Jembatan Existing
Sumber : BBPJN VIII



Gambar 3.4 Tampak Melintnag Jembatan Existing
Sumber : BBPJN VIII



Gambar 3.5 Tampak Memanjang Jembatan Rencana



Gambar 3.6 Tampak Melintang Jembatan Rencana

- **Literatur yang digunakan :**
  1. E-book Cable Stayed Bridge Second Edition (Walther 1999)
  2. SNI 1725 – 2016 tentang persyaratan Pembebanan Jembatan

3. Peraturan Kementrian PU "Pedoman Perencanaan Teknis Jembatan Beruji Kabel 2015"
4. SNI 2833-2016 tentang Peraturan Gempa pada Jembatan
5. E-book Design of Prestressing (Tylin dan Burn, 1998)

3.3 Preliminary Desain

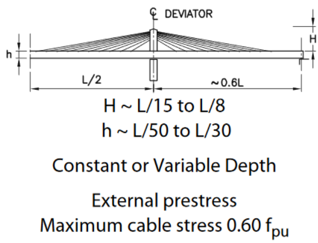
3.3.1 Susunan Kabel

Susunan kabel untuk arah melintang berupa *double plane system*, sedangkan untuk susunan arah memanjang seperti pada berupa *semi-harp pattern*. Berikut penjelasan untuk desain :

- Jarak kabel pada gelagar (Pedoman Perencanaan Teknis Jembatan 2015):

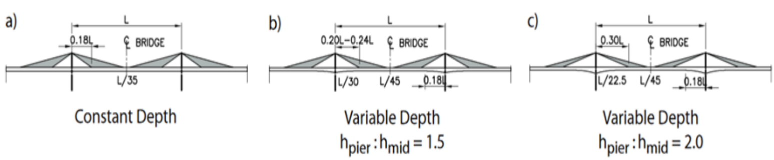
  - Jika berupa gelagar baja maka jaraknya (15m s.d. 25m)
  - Jika berupa gelagar beton maka jaraknya (5m s.d. 10m)

  Berikut adalah gambar spesifikasi desain jembatan extradosed yaitu pada gambar 3.7



Gambar 3.7 Spesifikasi Desain
Sumber : Mermigas, 2008

Chi chio menemukan bahwa offset kabel pertama 0,18 hingga 0,30 tergantung ketebalan girder.Berikut adalah gabar spesifikasi desain pada jembatan *extradosed* pada gambar 3.8 :



Gambar 3.8 Offset Kabel Pertama
Sumber : Mermigas, 2008

3.3.2 Dimensi Kabel

Menurut Troitsky, kabel digunakan dalam satu helai terdiri dari 7 kawat, untuk dimensi awal kabel dapat diperkirakan menggunakan persamaan 3.1 hingga 3.4 berikut :

- Mencari Gaya Kabel Utama

$$Ng,i \quad = \frac{R_{g,i}}{\sin \alpha_i} \tag{3.1}$$

Dimana :
$N_{g,i}$: gaya kabel utama
$R_{g,I}$: reaksi perletakan bentang sederhana akibat beban
α_i : sudut kemiringan kabel terhadap horizontal

- Mencari Tegangan Ijin Kabel

$$\sigma g \quad = \sigma ijin \left(\frac{g}{(g+q)}\right) \tag{3.2}$$

Dimana :
σ_g : tegangan ijin kabel akibat berat sendiri dan beban tetap
σ_{ijin} : tegangan ijin kabel akibat beban total = 0,6 σ_{putus}
g : berat sendiri dan beban tetap
q : beban hidup merata

- Mencari Luasan Kabel

$$\mathrm{Ai} \quad = \frac{\alpha\ \mathrm{Ng,i}}{\sigma \mathrm{g}} \tag{3.3}$$

Dimana :
A_i : luas kabel utama
α : fraksi beban yang dipikul kabel ($0< \alpha<1$),
1 bila kabel sangat kaku
$N_{g,i}$: gaya kabel utama
σ_g : tegangan ijin kabel akibat berat sendiri dan beban tetap

Analisa jembatan extradose didasarkan pada analisa elastisitas baja. Asumsi awal yang digunakan adalah ketika menerima beban mati kabel tetap akan terlihat melengkung, oleh karena lengkungan akibat beban mati modulus elastis efektif (E_{eff}) mempunyai nilai yang lebih kecil dari modulus elastic kabel baja (E_0). Jika kabel disimulasikan sebagai elemen sebenarnya dalam komputer maka akan didapat nilai (E_{eff})<(E_0), hal ini seharusnya digunakan terutama untuk kondisi kabel dengan bentang yang sangat panjang. Nilai (E_{eff}) selain dipengaruhi oleh panjang dan gaya juga oleh berat sendiri kabel. Modulus elastis efektif dapat direncanakan dengan formula berikut (Walther dkk, 1999).

$$E_{eff} \quad = E_o \frac{1}{1 + \frac{\Upsilon^2 {L_h}^2 E_o}{12\sigma^3}} \tag{3.4}$$

Dimana:

E_{eff} : modulus elastisitas efektif kabel (N/mm^2)
E_0 : modulus elastisitas dari material kabel (N/mm^2)
Υ : berat jenis dari material kabel tetap (N/mm^3)
L_h : jarak titik gantung kabel (mm)
σ : tegangan tarik dalam kabel (N/mm^2)

3.3.3 Sudut Kabel

Dalam sebuah jembatan kabel komponen vertikal didukung dari gaya di kabel mengangkat girder terus menerus, sementara komponen horizontal didukung prestresses gelagar. Konfigurasi kabel dan ketinggian menara adalah dua faktor yang mempengaruhi kecenderungan kabel, dan karenanya tindakan kabel di dek. Ini dua faktor akan dibahas dalam kaitannya dengan jembatan extradosed. Sudut yang di sarankan dalam perencanaan jembatan extradose ini berkisar 9-16 derajat . sudut yang rendah akan memudahkan dalam pelaksanaan, tingkat kelelahan pada kabel akan rendah di bandingkan dengan jembatan kabel stay. (chio chio 2010) untuk lebih jelasnya lihat Gambar 2.5 berikut :

3.3.4 Dimensi Struktur Pylon

Perencanaan untuk tinggi *pylon* dapat dilakukan dengan membandingkan antara tinggi rencana (H) dengan panjang bentang *deck* (L) dengan range nilai L/8-L/15 **Kris Mermigas (2008).**

Dimensi struktur *pylon* dapat dihitung dengan persamaan 3.5 berikut.

$$A_{perlu} = \frac{\sum P_{mi}}{f'c} \quad (3.5)$$

Kekuatan rencana dari kolom harus ditentukan dari kemampuannya menahan gaya aksial dan momen lentur akibat beban rencana dan momen lentur tambahan akibat pengaruh kelangsingan. Dalam hal ini, kekuatan rencana dihitung berdasarkan kekuatan nominal yang dikalikan dengan faktor reduksi kekuatan, dan telah memperhitungkan momen lentur tambahan akibat pengaruh kelangsingan. Pasal 5.7.2 SNI T 12 2004.

Kolom harus dikelompokkan sebagai tidak bergoyang atau bergoyang. Kolom tak bergoyang harus direncanakan menurut pasal 5.7.6.1, sedangkan kolom bergoyang harus direncanakan menurut pasal 5.7.6.2.

Beberapa hal yang perlu dihitung dalam perencanaan kolom :

a. Gaya tekan aksial (SNI 2847 2013, Pasal 21.6.1)

$$P_u \geq A_g \frac{f'c}{10} \quad (3.6)$$

Dimana dimensi kolom terpendek lebih besar dari 300 mm dan rasio dimensi b/h lebih besar dari 0,4.

b. Penulangan memanjang

$$\text{Ø}P_n\text{maks} = 0{,}8.\text{Ø}.(0{,}85.f'c.(A_g - A_{st})) + (f_y.A_{st}) \quad (3.7)$$

c. Persyaratan terhadap gaya geser

$$V_e = \frac{2.M_{pr}}{h} \quad (3.8)$$

$$V_u = \frac{M_{pr1} + M_{pr2}}{ln} \quad (3.9)$$

d. Pengekang kolom (SNI 2847 2013, Pasal 21.6.4.4)

$$A_{sh} = 0{,}3.\frac{\mathrm{s.h.c.f'c}}{fyt}.\left(\frac{A_g}{A_{oh}} - 1\right) \quad (3.10)$$

$$V_c = \left(1 + \frac{N_u}{14A_g}\right).\frac{\sqrt{\mathrm{f'c}}}{6}.bw.d \quad (3.11)$$

$$V_s = \frac{A_s.f_y.d}{s} \quad (3.12)$$

e. Panjang lewatan pada sambungan tulangan kolom

$$ld = \left(\frac{f_y.\Psi_t.\Psi_e..\Psi_s}{1{,}1.\lambda.\sqrt{f'c.\left(\frac{C_b + K_{tr}}{db}\right)}} - 1\right) \quad (3.13)$$

3.3.5 Ketinggian Lantai Kendaraan

Perencanaan untuk ketinggian deck dapat dilakukan dengan membandingkan antara tinggi rencana (h) dengan panjang bentang *deck* (L) dengan range nilai L/30-L/50 (Mermigas, 2008).

3.3.6 Perhitungan Gaya Prategang

Gaya Prategang dipengaruhi oleh momen total yang terjadi. Gaya prategang yang disalurkan harus memenuhi kontrol batas pada saat kritis. Persamaan 3.14 berikut menjelaskan hubungan antara momen total dengan gaya prategang (T.Y Lin, 1988).

$$F = \frac{M_t}{0{,}65\ h} \quad (3.14)$$

Dimana :

M_t : Momen total
h : tinggi balok

a. Tegangan ijin beton saat penyaluran gaya prategang

- Tegangan tekan

$$\sigma_{ci} = 0{,}6\ f'c \quad (3.15)$$

- Tegangan tarik

$$\sigma_{ti} = 0{,}25\sqrt{f'c} \tag{3.16}$$

b. Tegangan ijin beton setelah kehilangan gaya prategang

- Tegangan tekan

$$\sigma_{ci} = 0{,}45\ f'c \tag{3.17}$$

- Tegangan tarik

$$\sigma_{ti} = 0{,}5\sqrt{f'c} \tag{3.18}$$

3.3.7 Kehilangan Gaya Prategang

Kehilangan gaya prategang disebabkan oleh beberapa factor, antara lain (Lin dan Burn, 1998) :

1. Kehilangan langsung (*Immedietly Loss*), yaitu kehilangan gaya prategang yang terjadi setelah peralihan gaya prategang yang meliputi :

   a. Kehilangan prategang akibat perpendekan elastis

$$ES = K_{ES}.E_S.\frac{f_{cir}}{E_{ci}} \tag{3.19}$$

   Dimana :

   f_{cir} : tegangan beton akibat gaya prategang efektif
   E_s : modulus elastisitas tendon prategang
   E_{ci} : modulus elastisitas beton
   K_{ES} : 1,0 untuk struktur pratarik dan 0,5 untuk struktur pasca Tarik

   b. Kehilangan prategang akibat gesekan kabel

$$Ps = Px.e^{-(\mu\alpha+KL)} \tag{3.20}$$

   Bila ($\mu\alpha$+Kλx) tidak lebih besar dari 0,3 maka pengaruh kehilangan akibat friksi boleh dihitung sebagai berikut :

$$Ps = Px.(1+\mu\alpha+KL) \tag{3.21}$$

   Dimana :

K : koefisien wobble
L : panjang kabel
μ : koefisien friksi
α : sudut kelengkungan kabel

Berikut adalah nilai koefisien wobble dan koefisien kelengkungan (Tabel 3.1)

Tabel 3.1 Nilai Koefesien Wobble Dan Kelengkungan

Tipe Tendon		Koef. Wobble K per meter	Koef. Kelengkungan
Tendon pada selubung logam fleksibel	Tendon kawat	0,0033-0,0049	0,15-0,25
	Batang kekuatan tinggi	0,0003-0,002	0,08-0,30
	Strand 7 kawat	0,0016-0,0066	0,015-0,25
Tendon pada selubung logam kaku	Strand 7 kawat	0,0007	0,15-0,25
Tendon yang diminyaki terlebih dahulu	Tendon kawat dan strand 7 kawat	0,001-0,0066	0,05-0,15
Tendon yang diberi lapisan mastik	Tendon kawat dan strand 7 kawat	0,0033-0,0066	0,05-0,15

Sumber : Tylin dan Burn, 1998

c. Kehilangan prategang akibat slip angkur

$$\mathrm{ANC} = \Delta fs = \frac{\Delta_{\alpha}.E_s}{L} \tag{3.22}$$

Dimana :
Δ_α : deformasi pengangkuran
E_s : modulus elastisitas angkur
L : panjang total kabel

2. Kehilangan tak langsung (*Time Dependent Loss*), yaitu kehilangan gaya prategang yang bergantung pada fungsi waktu yang meliputi :

   a. Kehilangan prategang akibat rangkak beton (*creep*)

$$CR = K_{cr}.\frac{E_s}{E_c}.(f_{cir} - f_{cds}) \tag{3.23}$$

Dimana :
K_{cr} : 2,0 untuk struktur pratarik dan 1,6 untuk struktur pasca Tarik
E_s : modulus elastisitas tendon
E_c : modulus elastisitas beton
f_{cir} : tegangan beton akibat gaya prategang efektif setelah gaya prategang diberikan
f_{cds} : tegangan beton pada titik berat tendon akibat seluruh beban mati yang bekerja pada komponen struktur setelah diberi gaya prategang.

   b. Kehilangan prategang akibat susut beton (*shrinkage*)

$$SH = 8{,}2.10^{-6}.K_{sh}.E_s.(1 - 0{,}06\frac{V}{s}).(100 - RH) \tag{3.24}$$

Dimana :
K_{sh} : koefisien factor susut
V : volume beton
s : luas selimut yang terpapar udara
RH : kelembaban udara

Tabel 3.2 Nilai koefisien factor susut

KSH	Waktu Akhir perawatan hingga pemberian gaya prategang
0,92	1
0,85	3

0,8	5
0,77	7
0,73	10
0,64	20
0,58	30
0,45	60

Sumber : Tylin dan Burn, 1998

c. **Kehilangan prategang akibat relaksasi baja (*relaxation*)**

$$RE = [K_{re} - J.(SH + CR + ES)].C \qquad (3.25)$$

Berikut adalah nilai K_{re} dan J. Untuk lebih jelasnya lihat tabel 3.3 :

Tabel 3.3 Nilai K_{re} dan J

	Type of tendon	**Kre (Mpa)**	**J**
1	Strand/kawat stress-relieved 1860 MPa	138	0,15
2	Strand/kawat stress-relieved 1720 MPa	128	0,14
3	Kawat stress-relieved 1655 & 1620 MPa	121	0,13
4	Strand relaksasi rendah 1860 MPa	35	0,040
5	Kawat relaksasi rendah 1720 MPa	32	0,037
6	Kawat relaksasi rendah 1655 & 1620 MPa	30	0,035
7	Batabg stress-relieved 1000 & 1100 MPa	41	0,05

Sumber : Tylin dan Burn, 1998

Berikut adalah nilai C. Untuk lebih jelasnya lihat table 3.4 :

Tabel 3.4 Nilai C

fpi/fpu	Stress relieved strand or wire (C)	Stress-relieved bar or low relaxation strand or wire
0,80		1,28
0,79		1,22
0,78		1,16
0,77		1,11
0,76		1,05
0,75	1,45	1,00
0,74	1,36	0,95
0,73	1,27	0,90
0,72	1,18	0,85
0,71	1,09	0,80
0,70	1,00	0,75
0,69	0,94	0,70
0,68	0,89	0,66
0,67	0,83	0,61
0,66	0,78	0,57
0,65	0,73	0,53
0,64	0,68	0,49
0,63	0,63	0,45
0,62	0,58	0,41
0,61	0,53	0,37
0,60	0,49	0,33

Sumber : Tylin dan Burn, 1998

3.4 Pembebanan

Analisis jembatan Extradose terdiri dari analisis statik dan dinamik, hal ini dimaksudkan untuk menentukan variasi gaya pada elemen pendukung beban (gelagar, *pylon*, dan kabel), sedangkan analisis dinamis digunakan untuk menentukan kestabilan struktur.

3.4.1 Pembebanan Statis

Pada pembebanan statis,beban dibagi menjadi dua diantaranya :

- Beban tetap
- Berat sendiri

Berat sendiri merupakan berat elemen bahan dari struktur, ditambah dengan elemen bahan non struktur yang dianggap tetap. Berikut merupakan berat isi dan kerapatan massa untuk berat sendiri dari beberapa bahan menurut **SNI 1725-2016 Pasal 7.1 tabel 2**. Berikut adalah nilai berat jenis untuk beban mati . untuk lebih jelasnya lihat tabel 3.5.

Tabel 3.5 Berat isi untuk beban mati

No	Bahan	Berat Isi (Kn/m3)	Kerapatan massa (Kg/m3)
1	Lapisan permukaan beraspal	22	2245
2	Besi tuang	71	7240
3	Timbunan tanah di padatkan	17,2	1755
4	Kerikil dipadatkan	18,8 -22,7	1920-2315
5	Beton aspal	22	2245
6	Betn ringan	12,25-19,6	1250-2000
7	Beton fc < 35 MPa	22-25	2320
	Beton 35 < fc < 105 MPa	22 : 0,022 fc	2240-2,29 fc
8	Baja	78,5	7850
9	Kayu	7,8	800
10	Kayu keras	11	1125

Sumber : SNI 1725 2016

Berikut merupakan faktor beban untuk berat sendiri dari beberapa bahan menurut **SNI 1725-2016 Pasal 7.2 tabel 3**.6 Berikut adalah nilai factor beban untuk beban mati . untuk lebih jelasnya lihat tabel 3.6.

Tabel 3.6 Faktor beban untuk berat sendiri

Tipe Bahan	Faktor Beban (Υ_{MS})			
	Keadaan Batas Layan (Υ^{s} MS)		Keadaan Batas Ultimit (Υ^{u} MS)	
	Bahan		Biasa	Terkurangi
Tetap	Baja	1	1,1	0,9
	Aluminium	1	1,1	0,9
	Beton Pracetak	1	1,2	0,85
	Beton dicor ditempat	1	1,3	0,75
	Kayu	1	1,4	0,7

Sumber : SNI 1725 2016

Beban mati tambahan merupakan berat seluruh bahan yang membentuk suatu beban pada struktur yang merupakan elemen non struktural, dan besarnya dapat berubah seiring dengan bertambahnya umur struktur. Berikut merupakan faktor beban untuk beban mati tambahan menurut **SNI 1725-2016 Pasal 7.3 tabel 4**. Berikut adalah Faktor beban. untuk lebih jelasnya lihat tabel 3.7.

Tabel 3.7 Nilai Faktor Beban

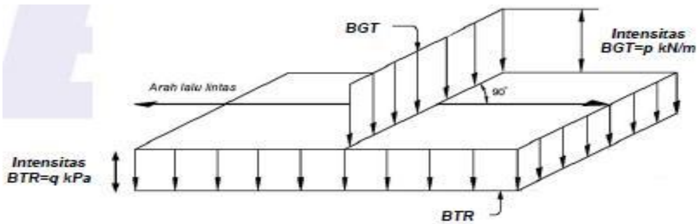
Tipe Bahan	Faktor Beban (Υ_{NA})			
	Keadaan Batas Layan (Υ^{s}_{NA})		Keadaan Batas Ultimit (Υ^{u}_{NA})	
	Bahan		Biasa	Terkurangi
Tetap	Umum	1,00 (1)	2,00	0,70
	Khusus Terawasi	1,00	1,40	0,80

Sumber : SNI 1725 2016

3.4.2 Beban Lalu Lintas

a. Beban Lajur "D"

Beban lajur "D" menurut **SNI 1725-2016 Pasal 8.3** terdiri dari beban terbagi rata (BTR) yang digabung dengan beban garis terpusat (BGT), lihat Gambar 3.9 berikut :



Gambar 3.9 Beban lajur D

Sumber : SNI 1725 2016

Beban terbagi rata (BTR) mempunyai intensitas q kPa, dengan besarnya q tergantung pada panjang total :

$$L \leq 30m \; ; \; q = 9{,}0 \text{ kPa} \qquad (3.26)$$

$$L \geq 30m \; ; \; q = 9{,}0\,(0{,}5+15/L) \text{ kPa} \qquad (3.27)$$

Panjang yang dibebankan L adalah panjang total BTR yang bekerja pada jembatan. Beban garis terpusat (BGT) dengan intensitas p KN/m harus ditempatkan tegak lurus arah lalu-lintas pada jembatan. Besarnya intensitas p adalah 49.0 kN/m. Berikut adalah nilai Faktor Beban D. untuk lebih jelasnya lihat tabel 3.8.

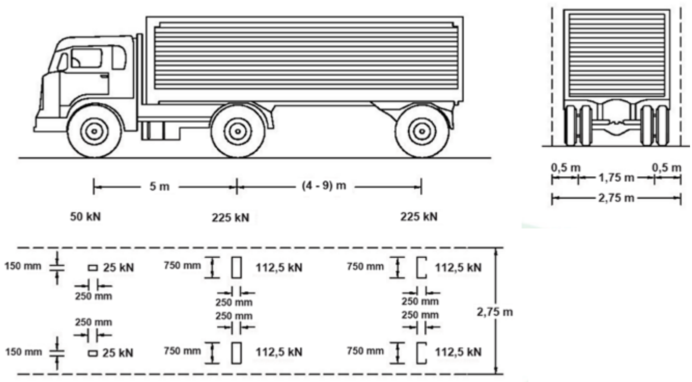
Tabel 3.8 Faktor beban untuk beban lajur D

Tipe Bahan	Faktor Beban (ϒ TD)		
	Bahan	Keadaan Batas Layan (Υs^s TD)	Keadaan Batas Ultimit (Υs^u TD)
Tetap	Beton	1	1,8
	Box Girder Baja	1	2

Sumber : SNI 1725 2016

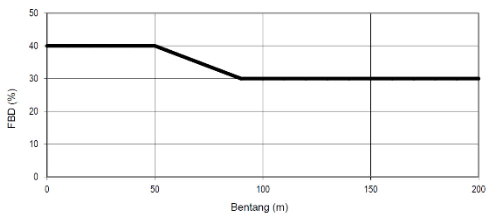
3.4.3 Beban Truck “T”

Pembebanan truk “T” menurut **SNI 1725-2016 Pasal 8.4** terdiri dari kendaraan truk semi trailer yang mempunyai susunan dan berat as seperti pada Gambar 3.5. Berat sendiri dari masing-masing as disebar menjadi dua beban merata sama besar yang merupakan bidang kontak antara roda dengan permukaan lantai . Jarak antara dua as tersebut antara 4 m sampai dengan 9 m, hal ini dikarenakan untuk mendapatkan pengaruh terbesar pada arah memanjang jembatan. Kendaraan truk “T” ini harus ditempatkan di tengah-tengah lajur lalu lintas rencana. Untuk lebih jelasnya lihat Gambar 3.10 berikut :



Gambar 3.10 Beban truck
Sumber : SNI 1725 2016

Berikut adalah nilai factor beban T. Untuk lebih jelasnya dapat dilihat pada table 3.9 dan factor beban dinamis gambar 3.11 :



Gambar 3.11 Faktor beban dinamis untuk beban T
Sumber : SNI 1725 2016

Tabel 3.9 Faktor beban untuk beban T

Tipe Bahan	Faktor Beban (ϒ TT)		
	Bahan	Keadaan Batas Layan (Υs^{s} TT)	Keadaan Batas Ultimit (Υs^{u} TT)
Tetap	Beton	1	1,8
	Box Girder Baja	1	2

Sumber : SNI 1725 2016

3.4.4 Gaya Rem

Gaya rem harus diambil yang terbesar dari :

- 25% dari berat gandar truk desain atau,
- 5% dari berat truk rencana ditambah beban lajur terbagi rata BTR.

Gaya rem tersebut harus ditempatkan di semua lajur rencana yang dimuati dan yang berisi lalu lintas dengan arah yang sama. Gaya ini harus diasumsikan bekerja horizontal diatas permukaan jalan pada masing-masing arah longitudinal dan dipilih yang paling menentukan.

3.4.5 Pembebanan Pejalan Kaki

Sesuai dengan peraturan SNI 1725:2016 8.9 semua elemen dari trotoar atau jembatan penyeberangan yang langsung memikul pejalan kaki harus direncanakan untuk beban nominal 5 kPa.

3.4.6 Beban Lingkungan

Beban lingkungan dapat terjadi karena pangaruh temperatur, angin, banjir, gempa, dan penyebab alamiah lainnya.

1. **Pengaruh susut dan rangkak**

   Pengaruh ini menurut **SNI 1725-2016 Pasal 9.3.2** dihitung menggunakan beban mati jembatan. Apabila rangkak dan penyusutan bisa mengurangi pengaruh muatan lainnya, maka nilai dari rangkak dan penyusutan tersebut harus diambil minimum (misalnya pada waktu transfer dari beton prategang). Berikut adalah nilai Faktor Beban. untuk lebih jelasnya lihat Tabel 3.10.

Tabel 3.10 Faktor beban akibat susut rangkak

Tipe Bahan	Faktor Beban (ϒ SH)	
	Keadaan Batas Layan (ϒs^{s} SH)	Keadaan Batas Ultimit (ϒs^{u} SH)
Tetap	1	0,5

Sumber : SNI 1725 2016

2. Pengaruh prategang

Menurut **SNI 1725-2016 Pasal 9.3.3** prategang akan menyebabkan pengaruh sekunder pada komponen-komponen yang terkekang pada bangunan statis tidak tentu. Pengaruh sekunder tersebut harus diperhitungkan baik pada batas daya layan ataupun batas ultimit. Prategang harus diperhitungkan sebelum (selama pelaksanaan) dan sesudah kehilangan tegangan dalam kombinasinya dengan beban-beban lainnya. Berikut adalah nilai Faktor Beban akibat prngaruh prategang. Untuk lebih jelasnya lihat Tabel 3.11.

Tabel 3.11 Faktor beban akibat gaya prategang

Tipe Bahan	Faktor Beban (ϒ PR)	
	Keadaan Batas Layan (ϒs^{s} PR)	Keadaan Batas Ultimit (ϒs^{u} PR)
Tetap	1	1

Sumber : SNI 1725 2016

3. Beban angin

Tekanan angin menurut **SNI 1725-2016 Pasal 9.6.1** yang ditentukan untuk angin horizontal diasumsikan disebabkan oleh angin rencana dengan kecepatan dasar (VB) sebesar 90 hingga 126 km/jam.

Untuk jembatan atau bagian jembatan dengan elevasi lebih tinggi dari 10 m di atas permukaan tanah atau permukaan air, kecepatan angin rencana V_{DZ} harus dihitung dengan persamaan berikut:

$$V_{DZ} = 2{,}5\ V_0 \left(\frac{V_{10}}{V_B}\right) \ln\left(\frac{Z}{Z_o}\right) \quad (3.28)$$

Dimana :

V_{DZ} : kecepatan angin pada elvasi rencana (km/jam)

V_0 : kecepatan gesekan angin, yang merupakan karakteristik meterologi, sebagaimana ditentukan dalam table 3.12, untuk berbagai macam tipe permukaan di hulu jembatan (km/jam)

V_{10} : kecepatan angin pada elevasi 10 m di atas permukaan tanah atau di atas permukaan air rencana (km/jam)

V_B : keccepatan angin rencana yaitu 90 hingga 126 km/jam

Z : elevasi struktur diukur dari permukaan tanah atau dari permukaan air dimana beban angin dihitung

Z_0 : panjang gesekan di hulu jembatan, yang merupakan karakteristik meterologi, ditentukan table 3.12 (mm)

Tabel 3.12 Nilai V_0 dan Z_0

Kondisi	Lahan Terbuka	Sub Urban	Kota
Vo (km/jam)	13,2	17,6	19,3
Zo (mm)	70	1000	2500

Sumber : SNI 1725 2016

a. Beban angin pada struktur (EWs)

Tekanan angin rencana (MPa) dapat ditetapkan menggunakan persamaan sebagai berikut :

$$PD = PB \left(\frac{V_{DZ}}{V_B}\right)^2 \quad (3.29)$$

Dimana :

P_B : tekanan angin dasar seperti ditentukan dalam table 3.11 (MPa)

Berikut adalah nilai Tekanan angin dasar. Untuk lebih jelasnya lihat Tabel 3.13.

Tabel 3.13 Tekanan angin dasar

Komponen Bangunan Atas	Angin Tekan (Mpa)	Angin Hisap (Mpa)
Rangka , Kolom , dan Pelengkung	0,0024	0,0012
Balok	0,0024	N/A
Permukaan dasar	0,0019	N/A

Sumber : SNI 1725 2016

Gaya total beban angin tidak boleh diambil kurang dari 4,4 kN/mm pada bidang tekan dan 2,2 kN/mm pada bidang hisap pada struktur rangka dan pelengkung, serta tidak kurang dari 4,4 kN/mm pada balok atau gelagar.

b. Beban angin pada kendaraan (EW_I)

Jembatan harus direncanakan memikul gaya akibat tekanan angin pada kendaraan, di mana tekanan tersebut harus diasumsikan sebagai tekanan menerus sebesar 1,46 N/mm, tegak lurus dan bekerja 1800 mm di atas permukaan jalan. Komponen beban angin yang bekerja tegak lurus maupun paralel terhadap kendaraan untuk beragai sudut serang dapat diambil sesuai dalam tabel 3.14 di mana sudut serang ditentukan tegak lurus terhadap arah permukaan kendaraan.

Tabel 3.14 Komponen Beban Angin Yang Bekerja

Sudut	Komponen tegak lurus	Komponen sejajar
derajat	N/mm	N/mm
0	1,46	0,00
15	1,28	0,18
30	1,20	0,35
45	0,96	0,47
60	0,50	0,55

Sumber : SNI 1725 2016

3.4.7 Pengaruh Beban Gempa

Menurut **SNI 2833-2016 Pasal 5** menyatakan bahwa jembatan harus direncanakan agar memiliki kemungkinan kecil untuk runtuh namun dapat mengalami kerusakan yang signifikan dan gangguan terhadap pelayanan akibat gempa.

Beban gempa diambil sebagai beban horizontal yang ditentukan berdasarkan perkalian antara koefisien respons elastik (C_{sm}) dengan berat struktur ekuivalen yang kemudian dimodifikasi dengan faktor modifikasi respons (R) dengan formulasi sebagai berikut:

$$EQ = \frac{C_{sm}}{R_d} x\, Wt \qquad (3.30)$$

Dimana :

E_Q : gaya gempa horizontal statis (kN)
C_{sm} : koefisien respons gempa elastis
R : faktor modifikasi respons
W_t : berat struktur terdiri dari beban mati dan hidup (kN)

Koefisien respons elastik C_{sm} diperoleh dari peta percepatan batuan dasar dan spektra percepatan sesuai daerah gempa dan periode ulang gempa rencana. Koefisien percepatan yang diperoleh berdasarkan peta gempa dikalikan dengan suatau faktor amplifikasi sesuai dengan keadaan tanah sampai kedalaman 30 meter di bawah struktur jembatan.

3.5 Kombinasi Beban

Untuk jembatan beruji kabel yang memiliki tiga bentangan, kombinasi beban hidup mengikuti , tujuh konfigurasi utama yaitu pada Tabel 3.15 berikut :

Tabel 3.15 Kombinasi Beban

Kode	Kombinasi
Kuat I	1,3MS+2MA+1,8TD+1,8TP+1EUn
Kuat III	1,3MS+2MA+1,4Ews+1Eun
Kuat IV	1,3MS+2MA+1Eun
Kuat V	1,3MS+2MA+0,4Ews+1EW1+1Eun

Ekstrem 1	1,3MS+2MA+0,5TD+0,5TB+0,5TP+0,3Eqx+1Eqy
Ekstrem 2	1,3MS+2MA+0,5TD+0,5TB+0,5TP+1Eqx+0,3Eqy
Daya layan 1	1MS+1MA+1TD+1TB+1TP+0,3Ews+EW1+1Eun
Daya layan 2	1MS+1MA+1,3TD+1,3TB+1,3TP+1Eun
Daya layan 3	1MS+1MA+0,8TD+0,8TB+0,8TP+Eun
Daya layan 4	1MS+1MA+1TD+0,7Ews+1Eun

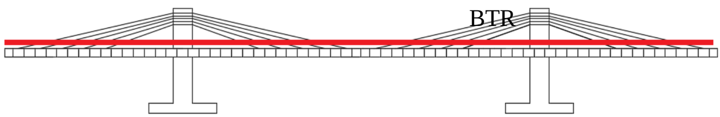
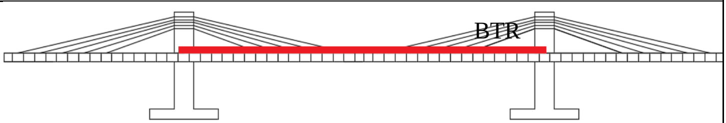
Sumber : SNI 1725 2016

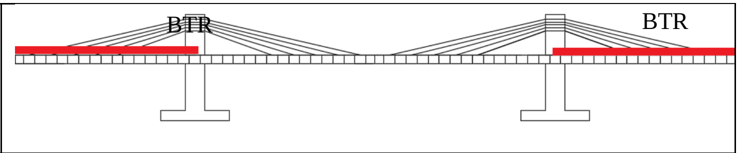
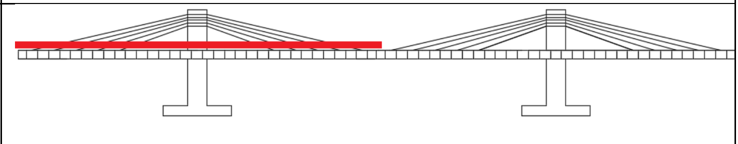
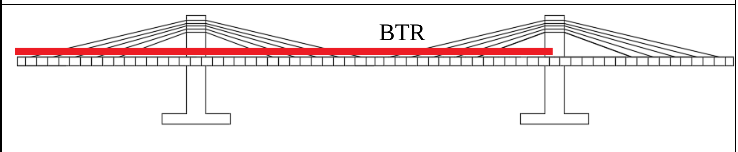
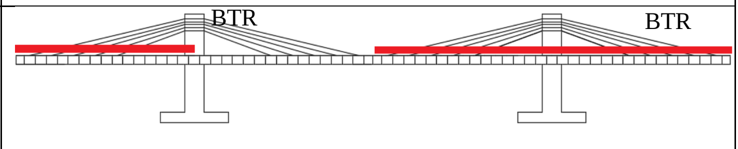
Dimana :

MS : Beban mati komponen structural dan non struktural
MA : Beban mati perkerasan dan utilitas
TB : Gaya akibat rem
EQ : Gaya akibat gempa
TD : Beban lajur "D"
TT : Beban lajur "T"
TP : Beban pejalan kaki
E_{Un} : Gaya akibat temperature seragam
E_{Ws} : Beban angin pada struktur
E_{W1} : Beban angin pada kendaraan

Berikut adalah konfigurasi beban yang di gunakan pada jembatan extradosed ini yaitu dapat dilihat pada Tabel 3.16.

Tabel 3.16 Konfigurasi Beban BTR

	Beban
1	
2	

3	
4	
5	
6	
7	

3.6 Kontrol Stabilitas Aerodinamis

Perilaku aerodinamis akibat angin terhadap jembatan *extradosed* perlu untuk dianalisa dan dikontrol, karena perilaku ini merupakan salah satu penyebab terjadinya kegagalan struktur. Analisa stabilitas pada desain ini meliputi *vortex-shedding* (tumpahan pusaran angin) dan *flutter* (efek ayunan). Akan tetapi dalam menganalisa efek angin yang bekerja pada jembatan seperti desain ini, sebenarnya perlu juga adanya *wind tunnel test* menggunakan model.

3.6.1 Frekuensi Alami

Menurut Walther, 1999 hal ini dapat dihitung dengan frekwensi lentur balok (fB) dan frekuensi alam akibat torsi (fT) yang didekati menggunakan persamaan berikut ini :

$$fB = \frac{1,1}{2\pi}\left(\frac{g}{v_{maks}}\right)^{0,5} \quad (3.31)$$

$$fT = \frac{\bar{b}}{2r} fB \quad (3.32)$$

Dimana:

f_B : frekuensi alami lentur balok (Hz)
g : percepatan gravitasi = 9,81 (m/s^2)
V_{maks} : deformasi statis maksimum akibat berat sendiri (m)
f_T : frekuensi alami torsi balok (Hz)
$\bar{b}$: jarak kabel arah melintang (m)
r : jari-jari girasi penampang lantai kendaraan (m)

3.6.2 Efek Vortex-Shedding

Menurut Walther, 1999 pada kecepatan angin tertentu yang disebut dengan kecepatan kritis, akan terjadi pusaran angin (vortex-shedding). Untuk memperoleh nilai percepatan kritis tersebut, digunakan persamaan 3.33 angka Strouhal (S).

$$S = \frac{f_B . h}{V} \quad (3.33)$$

Dimana:

S : angka *Strouhal*
h : tinggi lantai kendaraan
V : kecepatan angin dihitung berdasarkan angka *Strouhal*

Selanjutnya dilakukan evaluasi efek pusaran dengan angka *Reynold* (Re). Akibat kecepatan angin yang bekerja besarnya angka *Reynold* harus memenuhi persyaratan, nilai *Re* harus berkisar antara $10^5 – 10^7$. Berikut persamaan untuk angka *Reynold.*

$$\text{Re} = \frac{\text{VB}}{\bar{v}} \quad (3.34)$$

Dimana:

Re : Angka *Reynold*
B : Lebar lantai kendaraan
$\bar{v}$: Viskositas kinematik udara (0,15 cm^2/dt)

Akibat adanya terpaan angin, akan terjadi gaya angkat (*uplift*) yang besarnya dapat dihitung dengan persamaan 3.35 berikut :

$$Fo = \rho \frac{V^2}{2} Ch \tag{3.35}$$

Dimana:

Fo : Gaya angkat

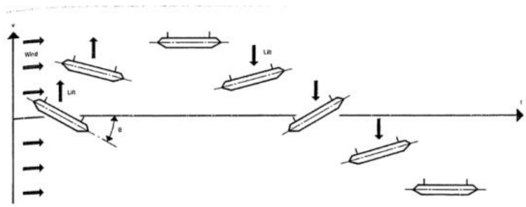
ρ : Berat volume udara (1,3 kg/m^3)

C : Koefisien gaya angkat lantai kendaraan

h : Tinggi lantai kendaraan

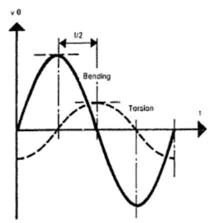
3.6.3 Efek *Flutter* (Efek Ayun)

Fenomena efek ayunan yang terjadi pada kecepatan kritis akan menimbulkan ayunan lentur (f_B) dan ayunan torsi (f_T), yang harus dihindari adalah nilai amplitudo akibat ayunan lentur dan ayunan torsi tidak terjadi secara bersamaan. Maka dari itu desain yang ideal, nilai perbandingan dari keduanya sebaiknya memiliki perbedaan fase sebesar π/2 atau berkisar 1,57 detik. Untuk lebih jelasnya lihat ilustrasi pada Gambar 3.12 dan 3.13 berikut ini.



Gambar 3.12 Representasi sederhana *flutter*

Sumber : Walther dkk, 1999



Gambar 3.13 Efek ayunan dengan beda fase π/2

Sumber : Walther dkk, 1999

3.7 Kontrol Stabilitas

Analisa permodelan stabilitas aerodinamis jembatan *cable-stayed* menggunakan program bantu MIDAS/Civil.

3.8 Pengerjaan Gambar

Tahapan akhir dari perencanaan berupa penyusunan gambar kerja, dalam pengerjaannya digunakan program bantu AutoCAD.

BAB IV
HASIL DAN PEMBAHASAN

4.1 Preliminary Desain

Sebelum melakukan perhitungan struktur sekunder maupun primer perlu dilakukan perkiraan dimensi awal berdasarkan referensi yang diperlukan dengan menyesuaikan pada parameter yang ada. Dimensi awal yang perlu diperkirakan antara lain meliputi susunan kabel, dimensi gelagar, dimensi kabel dan *pylon*, yang kemudian akan digunakan sebagai data awal dalam analisa struktur. Jika ternyata dalam analisa diketahui kemampuan struktur tidak memenuhi syarat, maka perlu dilakukan perubahan pada parameter yang telah ditentukan.

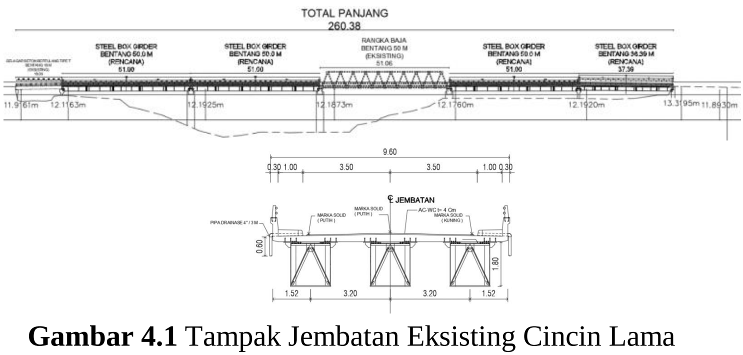
4.1.1 Preliminary Geometrik Jembatan

Dalam menentukan preliminary geometrik jembatan harus mempertimbangkan kondisi eksisting jembatan cincin lama widang.

a. Kondisi eksisting jembatan

Panjang total jembatan	:	260,5 m
Jarak antar pylon	:	37,4 + 51,2 + 51,2 + 51,2 + 51,2 + 18,3
Lebar jembatan	:	1 + 7 + 1

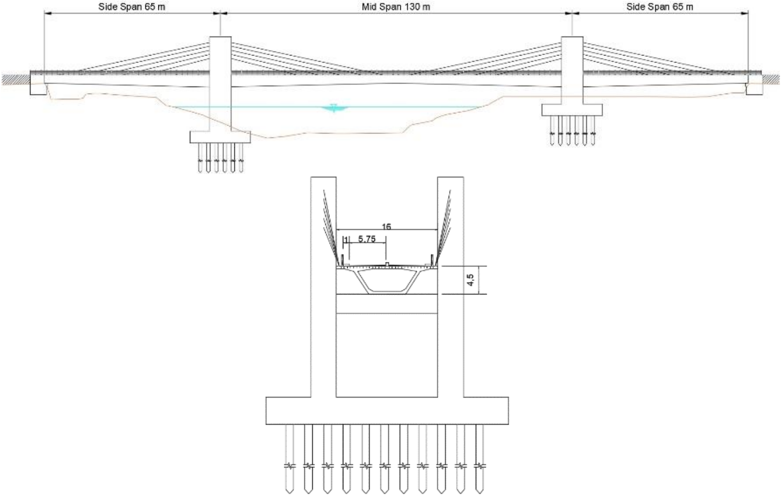
Layout jembatan eksisting cincin lama widang tertera pada Gambar 4.1



Gambar 4.1 Tampak Jembatan Eksisting Cincin Lama

b. Perencanaan layout jembatan

Dengan kondisi eksisting jembatan seperti yang tertera pada Gambar 4.1 maka direncanakan jembatan *extradosed* dengan bentang tengah sepanjang 130 m dan bentang samping jembatan sepanjang 65 m. Sedangkan untuk lebar jembatan direncanakan menjadi 14 m agar jembatan tidak terlalu langsing dan dapat menahan puntir jembatan. Untuk lebih jelasnya dapat dilihat pada Gambar 4.2.



Gambar 4.2 Tampak Jembatan Rencana

4.1.2 Konfigurasi Susunan Kabel

Konfigurasi susunan kabel pada arah melintang berupa *Double Planes System*, sedangkan untuk arah memanjang menggunakan *Semi Harp Pattern.*

a. Jarak kabel pada gelagar menurut Whalter dkk. (1999) :
- Bila menggunakan dek beton maka jarak kabel 5 m - 10 m
- Bila menggunakan dek baja maka jarak kabel 15 m - 25 m

Pada desain ini digunakan lantai kendaraan dari beton sehingga jarak antar kabel pada gelagar sebesar 5 m.

b. Perencanaan untuk tinggi *pylon* dapat dilakukan dengan membandingkan antara tinggi rencana (H) dengan panjang bentang *deck* (L) dengan range nilai L/8-L/15 **Kris Mermigas (2008)**.

Direncanakan (H/L)	= 1/10
Bentang jembatan L	= 130 m
Maka diperoleh H	= 130 m x 1/10
	= 13 m

c. Menurut Chio cho (2000) Offset kabel pertama untuk jembatan *extradosed* berkisar 0,18 hingga 0,30. Pada perencanaan kali ini offset kabel pertama jembatan *exradosed* di rencanakan sebagai berikut :

Direncanakan (0,22 L)	= 0,22 x 130
	= 28,6 m
Digunakan	= 28 m

4.1.3 Dimensi Lantai Kendaraan

Perencanaan untuk ketinggian dari lantai kendaraan dapat direncanakan dengan membandingkan tinggi rencana (h) dengan jarak antar pylon (L) dengan range nilai L/30 – L/50 **Kris Mermigas (2008)**.

a. Lantai kendaraan dekat pylon

h/L	= 1/30
L	= 130 m
h	= 130 m x 1/30
	= 4,33 m
h digunakan	= 4,5 m

b. Lantai kendaraan pada tengah bentang

h/L	= 1/45
L	= 130 m
h	= 130 m x 1/45
	= 2,88 m
h digunakan	= 3 m

4.1.4 Dimensi Kabel dan Angkur

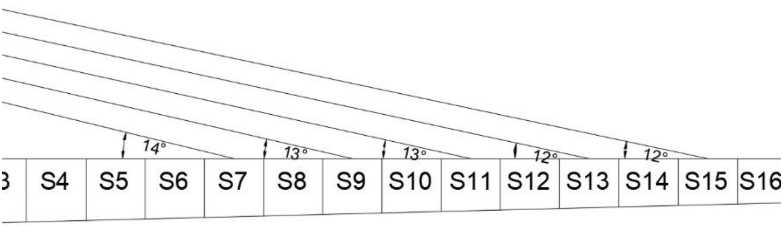
Berdasarkan RSNI-03-2005 pasal 12.6 kabel pemikul utama yang dipergunakan pada jembatan kabel dan jembatan gantung harus dibuat dari material mutu tinggi dengan kuat Tarik minimum 1800 N/mm^2.

Ada dua jenis kabel pararel *VSL 7-wire strand* yang dapat digunakan untuk jembatan kabel, lihat tabel 4.1.

Tabel 4.1 Jenis Kabel dan Angker

Standart	ASTM A 416-06 Grade 270	Euronorme 138-3
Ø (mm)	15,2	15,7
A_s (mm^2)	140	150
f_u (f_{ijin}=0,7 f_u) (MPa)	1860 (1302)	1770 (1239)
Ukuran angker	7, 12, 19, 31, 37, 61, 91, 109, dan 127 strand	

Berikut adalah gambar sudut kabel dari sisinan kabel jembatan *extradosed* yang ditunjukan pada Gambar 4.3 berikut.



Gambar 4.3 Susunan sudut kabel

Kemiringan kabel pada jembatan bertipe *extradosed* berkisar 9°-16° menurut Mermigas (2008). Untuk perencanaan jembatan *extradosed* ini memiliki kemiringan sudut pada kabel pertama dan seterusnya, berturut-turut adalah 14°, 13°, 13°, 12°,

12^o. Sedangkan untuk offset kabel pertama berjarak 28 m terhadap pylon.

Dimensi awal kabel didekatkan dengan persamaan berikut (Gimsing, 1983)

$$Asc = \frac{(W\lambda + P)\cos\theta}{\frac{(0{,}8 f_u)\sin 2\theta}{2} - \gamma . a}$$

Dimana :

Asc	=	Luas penampang kabel
W	=	Beban mati dan hidup merata
P	=	Beban terpusat
λ	=	Jarak antar angker kabel pada gelagar
θ	=	Sudut kabel terhadap horizontal
γ	=	Berat jenis kabel (77,01 kN/m^3)
f_u	=	Tegangan putus kabel (1860 MPa)
a	=	Jarak mendatar dari pylon ke angker kaber pada gelagar

Menghitung kebutuhan jumlah kabel dapat digunakan perhitungan sebagai berikut.

$$n = \frac{Asc}{As}$$

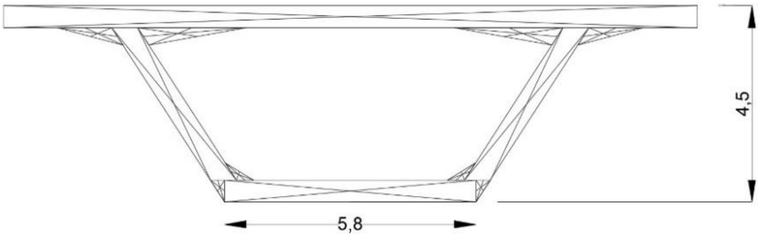
4.1.4.1 Beban Mati

Pada perencanaan jembatan menerus ini penampang girder dibuat seragam agar mempermudah dalam pelaksanaannya. Mengingat metode pelaksanaan yang digunakan adalah metode *balance cantilever* sehingga memerlukan keseimbangan pada setiap pemasangan girder itu sendiri. Untuk data perencaan girder sebagai berikut :

- Tinggi girder dekat pylon = 4,5 m
- Tinggi girder di tengah bentang = 3 m
- Ketebalan feln atas = 500 mm
- Ketebalan flen bawah = 500 mm

- Ketebalan web = 500 mm

Berikut adalah ilustrasi gambar penampang box girder untuk segmen 1,lebih jelasnya dapat melihat Gambar 4.4.



Gambar 4.4 Penampang box grider

Selanjutnya perhitungan inersia penampang box girder terhadap garis netral penampang.

Untuk mendapatkan garis netral penampang, menggunakan rumus sebagai berikut :

$$y = \frac{\sum(A_i y_i)}{\sum A_i}$$

Dimana :

A_i = Luas tiap bagian penampang box (mm)

y_i = Jarak garis berat tiap bagian penampang terhadap serat bawah penampang (mm)

Sedangkan untuk perhitungan inersia penampang sendiri dapat menggunakan rumus sebagai berikut.

- Untuk penampang persegi :

$$I_o = \frac{1}{12} bh^3$$

- Untuk penampang segitiga :

$$I_o = \frac{1}{36} bh^3$$

Untuk inersia penampang total box girder dapat dihitung menggunakan rumus berikut.

$$I = \sum(I_o + (A_i d_i)^2)$$

Dimana :

I_o = Inersia tiap-tiap bagian (m^4)

A_i = Luas tiap bagian penampang (m^2)

d_i = Jarak daris garis netral bagian penampang ke garis netral penampang keseluruhan c.g.c (m)

Dalam perhitungan penampang box girder dibagi menjadi beberapa bagian. Berikut contoh perhitungan penampang box girder pada segmen 1 seperti tertera pada Tabel 4.2

Tabel 4.2 Perhitungan penampang box girder

No.	Bentuk	n	b	h	Axn	y	Axy
1	Persegi	1	16	0,5	8	4,25	34
2	Segitiga	2	1,25	0,4	0,5	3,87	1,93
3	Segitiga	2	2,5	0,4	1	3,87	3,87
4	Persegi	1	0,59	3,5	2,087	2,25	4,69
5	Segitiga	2	0,4	0,4	0,16	0,63	0,11
6	Segitiga	2	0,325	0,5	0,163	0,33	0,05
7	Persegi	1	5,8	0,5	2,9	0,25	0,73
				A_{total}	14,81	$A.y_{total}$	45,38
				ya	1,436	yb	3,06

Keterangan :

- n = Jumlah bagian pada penampang
- b = Lebar tiap bagian (m)
- h = Tinggi tiap bagian (m)
- A = Luas tiap bagian (m^2)
- ya = Jarak cgc terhadap serat atas (m)
- yb = Jarak cgc terhadap serat bawah (m)

Berikut adalah contoh perhitungan terhadap beban mati box girder terhadap kabel pertama.

quD = (A segmen 6 + A segmen 7)/2 x Bj beton x Υ^{u} MS
= (14,61 + 14,57)/2 x 24kN/m^3 x 1,3
= 455,27 kN/m

4.1.4.2 Beban Mati Tambahan

Berikut adalah contoh perhitungan terhadap beban mati tambahan pada box girder terhadap kabel pertama.

a. Berat aspal

qu_{aspal} = tebal aspal x Bj aspal x lebar jalan x Υ^u MA
= 0,07m x 22kN/m^3 x 14 x 2
= 43,12 kN/m

b. Berat angker
Pu_{angker} = 5kN x Υ^u MA
= 10 kN

c. Berat trotoar
$qu_{trotoar}$ = tebal trotoar x Bj beton x lebar trotoar x Υ^u MA
= 0,25m x 24kN/m^3 x 1 x 2
= 12 kN/m

quSD = qu_{aspal} + $qu_{trotoar}$
= 43,12 + 12
= 55,12 kN/m

PuSD = Pu_{angker}
= 10 kN

4.1.4.3 Beban Hidup

Berikut adalah contoh perhitungan terhadap beban hidup pada box girder terhadap kabel pertama.

a. Beban terbagi rata (BTR)
Besar beban BTR adalah 9 kPa menurut SNI 1725-2016 pasal 8.3.1
qul_1 = BTR x lebar jalan x Υ^u TD
= 9kN/m^2 x 14 x 1,8
= 226,8 kN/m

b. Beban garis terpusat (BGT)
Besar beban BGT adalah 49kN/m menurut SNI 1725-2016 pasal 8.3.1
Pul_1 = BGT x lebar jembatan x Υ^u TD
= 49kN/m x 14 x 1,8
= 1234,8 kN

c. Beban truck (T)
Besar beban 1 As roda truck sebesar 225 kN menurut SNI 1725-2016 pasal 8.4.1
Pul_2 = T x jumlah lajur x Υ^u TT

= 225kN x 4 x 1,8
= 1620 kN

d. Beban pejalan kaki

Besar beban pejalan kaki adalah 5 kPa menurut SNI 1725-2016 pasal 8.9

qu_{TP} = TP x lebar trotoar x Υ^{u} TP
= 5 kN/m^2 x 1 x 1,8
= 9 kN/m

Berdasarkan hasil hasil perhitungan beban BGT dan BTR memiliki hasil lebih besar dari pada beban truck sehingga beban BGT dan BTR dipergunakan sebagai analisa beban hidup.

qul = qul_1 + qu_{TP}
= 226,8 + 9
= 235,8 kN/m

Pul = Pul_1
= 1234,8 kN

4.1.4.4 Perhitungan Beban yang dipikul Kabel (Wλ+P)

Beban yang dihitung meliputi beban hidup, beban mati, dan beban mati tambahan lain seperti beban pejalan kaki pada trotoar. Dimana jarak antar kabel di desain 8m, sehingga satu kabel memikul beban-beban di atas selebar 8m. Dan karena gelagar ditopang menggunakan 2 kabel, maka perhitungan beban yang dipikul dibagi dua. Berikut adalah contoh perhiutngan pada kabel pertama.

W = quD + quSD + qul
= 455,27 + 55,12 + 235,8
= 746,19 kN/m

P = PuSD + Pul
= 10 + 1234,8
= 1244,8 kN

Wλ+P = 746,19 x 8 + 1244,8
= 7214,32 kN

Dimana :

θ = 14^o

a = 28 m
σ_{ijin} = 0,6 x f_u
= 0,6 x 1860 MPa
= 1116 MPa
= 1116000 kN/m^2

$$Asc = \frac{(W\lambda+P)\cos\theta}{\frac{(0,6f_u)\sin 2\theta}{2} - \gamma.a}$$

$$= \frac{7214,32\ .\ \cos 14}{\frac{1116000\ .\ \sin 2x14}{2} - 77,01\ .\ 28}$$

= 0,026942 m^2
= 26942,86 mm^2

Dimensi kabel Ø15,2 mm ; As = 140 mm^2

Jumlah kabel satu sisi : n $= \frac{Asc}{2\ .\ As}$

$= \frac{26942,86}{2\ .\ 140}$

= 97 buah

Perhitungan selanjutnya akan ditabelkan pada tabel 4.3

Tabel 4.3 Jumlah Strand Pada Tendon

	W_D (kN/m)	W-W_D (kN/m)	λ (m)	P (kN)	θ	a (m)	Asc (mm)	n
1	455,27	290,92	8	1244,8	14	28	26942,86	97
2	452,91	290,92	8	1244,8	13	36	28990,35	104
3	450,62	290,92	8	1244,8	13	44	28990,47	104
4	448,41	290,92	8	1244,8	12	52	31409,81	113
5	446,76	290,92	9	1244,8	12	60	34684,23	124

Dalam pelaksanaan, kabel akan mengalami lendutan akibat beban kabel itu sendiri. Namun dalam analisa dianggap lurus dengan memberikan factor koreksi terhadap modulus elastisitas kabel tersebut (E_{eff}) menggunakan persamaan tersebut :

$$E_{eff} = \frac{E_o}{1+\left(\frac{\gamma^2.L^2.E_0}{12.\sigma^3}\right)}$$

Dimana :
E_{eff} = modulus elastisitas efektif
E_o = modulus elastisitas kabel (195 GPa)
γ = berat jenis kabel (77,01 kN/m^3)
L = panjang horizontal kabel
σ = tegangan tarik kabel
= 0,6 . f_u
= 1116 MPa

Perhitungan nilai modulus elastisitas efektif kabel dapat dilihat pada tabel 4.4.

Tabel 4.4 Modulus elastisitas efektif kabel

Kabel	L (m)	E_0 (MPa)	ϒ (kN/m^3)	σ (MPa)	Eeff (MPa)
1	28	195000	77,01	1116	194989,40
2	36		77,01		194982,48
3	44		77,01		194973,83
4	52		77,01		194963,45
5	60		77,01		194951,34

Dari tabel diatas dapat dilihat bahwa koreksi modulus elastisitas dari kabel sangatlah kecil kurang dari 0,05%, sehingga dapat diabaikan . Dengan kata lain lendutan akibat berat sendiri kabel sangatlah kecil, sehingga dapat dianggap sebagai kabel lurus.

4.1.5 Dimensi Pylon

Spesifikasi material *pylon* yang direncanakan adalah sebagai berikut :

- Material = beton bertulang
- f'c beton = 50 MPa
- fy tulangan = 400 MPa
- Tipe = Box

Preliminary struktur *pylon* ditentukan berdasarkan perbandingan dengan jembatan *extradosed* yang telah ada dengan Panjang bentang yang hamper sama. Oleh karena itu direncanakan dimensi *pylon* sebesar :

- b = 8 m
- h = 4 m
- flens = 1 m
- web = 1 m

4.2 Perencanaan Struktur Sekunder

Struktur sekunder pada jembatan *extradosed* ini berupa *railing* jembatan dan trotoar. Struktur sekunder sendiri tidak melalui tahap analisa bersama dengan struktur utama. Karena struktr sekunder tidak mempengaruhi perlaku terhadap struktur utama, sehinffa dapat dilakukan analisa secara terpisah.

4.2.1 Railing Jembatan

4.2.1.1 Perencanaan Railing

Pada perencanaan railing jembatan ini menggunakan profil baja. Dengan peraturan yang mengacau kepada SNI 1725-2016. Sedangkan untuk perencanaan beban mengacu kepada RSNI T-02-2005.

1. Spesifikasi *Railing*

   Railing jembatan ini menggunakan baja dengan mutu BJ 41, dengan spesifikasi sebagai berikut :

   Fy = 250 MPa
   Fu = 410 MPa
   E = 200000 MPa

2. Dimensi *Railing*

   a. Tiang sandaran

   Profil menggunakan *rectangular hollow* 200x200x8 setinggi 1,7 m dengan spesifikasi sebagai berikut :

   b = 200 mm　　A = 61,44 cm^2
   h = 200 mm　　Ix = 3781,4 cm^4
   t = 8 mm　　Iy = 3781,4 cm^4
   w = 46,57 kg/m

   b. Pipa horizontal

   Profil meggunakan *circular hollow* Ø3,5” sepanjang 2 m dengan spesifikasi sebagai berikut :

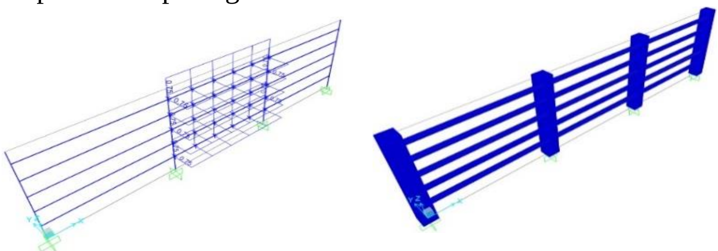
   D = 88,9 mm　　I = 96,3 cm^4

ts = 4 mm　　　　Z = 28,9 cm^3
w = 8,38 kg/m　　r = 3 cm
A = 10,7 cm^2

3. Perencanaan pembebanan *railing*
   Perencanaan pembebanan mengacu kepada RSNI T-02-2005 pasal 12.5. Beban yang bekerja pada penghalang lalu lintas harus direncanakan untuk menahan beban tumbukan rencana ultimit kearah vertical dan melintang yang besarnya w = 0,75 KN/m.

4.2.1.2 Permodelan Railing

1. Permodelan Struktur Railing
   Permodelan struktur menggunakan SAP2000 dilakukan menggunakan 3 segmen pada *railing*, dengan tiap-tiap segmen sepanjang 2 meter. Permodelan struktur dan beban dapat dilihat pada gambar 4.5.



Gambar 4.5 Permodelan railing jembatan

2. Hasil Analisa SAP2000
   Berikut adalah hasil rekapitulasi gaya dalam yang terjadi akibat beban tumbukan kendaraan pada *railing* yang dapat dilihat pada tabel 4.5.

Tabel 4.5 Rekapitulasi Gaya Dalam Railing

Nama Frame	Gaya Dalam					
	P (Kg)	V2 (Kg)	V3 (Kg)	M2 (Kg.m)	M3 (Kg.m)	Torsi (Kg.m)
Pipa	128,11	132,42	122,37	34,33	43,66	0
Tiang	812,84	104,45	540,84	617,26	2,6	163,67

4.2.1.3 Kontrol Kapasitas Pipa Railing

1. Kontrol Kapasitas Lentur

   Kelangsingan batang : [SNI 1729:2015 Tabel B4.1b]

   $\lambda = \frac{D}{t} = \frac{88,9}{4} = 22,225 \quad < \quad \lambda_p = 0,07\,\frac{E}{fy} = 0,07\,\frac{200000}{250} = 56$ **(Penampang kompak)**

   Momen nominal : [SNI 1729:2015 Pasal F8]

   ØMn = 0,9 . Zx . Fy
   = 0,9 x 28,9 x 2500
   = 65025 Kg.cm
   = 650,25 Kg.m > Mu = 43,66 Kg.m **(Memenuhi)**

2. Kontrol Kapasitas Tekan

   Kelangsingan batang : [SNI 1729:2015 Tabel B4.1a]

   $\lambda = \frac{D}{t} = \frac{88,9}{4} = 22,225 \quad < \quad \lambda_r = 0,11\,\frac{E}{fy} = 0,11\,\frac{200000}{250} = 88$ **(Memenuhi)**

   Tekan nominal : [SNI 1729:2015 Pasal E3]

   $\frac{KL}{r} = \frac{1\,.\,2000}{30} = 66,67 \quad < \quad 4,71\sqrt{\frac{E}{fy}} = 123,69$

   $Fe = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 200000}{(66,67)^2} = 444,132$

   $ØPn = 0,9\,.\left[0.685^{\frac{fy}{fe}}\right].\,fy\,.\,Ag$

   $= 0,9\,.\left[0.685^{\frac{250}{444,132}}\right].\,2500\,.\,10,7$

   = 19457,6 Kg > Pu = 128,11 Kg

3. Interaksi Lentur Aksial

   Kontrol interaksi lentur aksial [SNI 1729:2015 Pasal H1]

   $\frac{Pu}{ØPn} = \frac{128,11}{19457,6} = 0,0066 < 0,2$ (Rumus 2)

Jadi

$$\frac{Pu}{2\emptyset Pn} + \frac{Mux}{\emptyset Mnx} + \frac{Muy}{\emptyset Mny} = \frac{128,11}{2x19457,6} + \frac{43,66}{650,25} + \frac{34,33}{650,25}$$

= 0,123 < 1 **(Memenuhi)**

4.2.1.4 Kontrol Kapasitas Tiang Sandaran

1. Kontrol Kapasitas Lentur

Kontrol kelangsingan

Pelat Sayap

$\lambda = \frac{b}{t} = \frac{200}{8} = 25 < \lambda_p = 1,12\sqrt{\frac{E}{fy}} = 31,678$ **(Kompak)**

Pelat Badan

$\lambda = \frac{h}{t} = \frac{200}{8} = 25 < \lambda_p = 2,42\sqrt{\frac{E}{fy}} = 68,448$ **(Kompak)**

Momen nominal

ØMn = 0,9 . Zx . Fy
= 0,9 x 442,62 x 2500
= 995895 Kg.cm
= 9958,95 Kg.m > Mu = 617,26 Kg.m **(Memenuhi)**

2. Kontrol Kapasitas Tekan

Kelangsingan batang :

$\lambda = \frac{b}{t} = \frac{200}{8} = 25$ < $\lambda_r = 1,4\sqrt{\frac{E}{fy}} = 1,4\sqrt{\frac{200000}{250}} = 39,6$ **(Memenuhi)**

Tekan nominal

$\frac{KL}{r} = \frac{2,1 .170}{7,845} = 45,5 < 4,71\sqrt{\frac{E}{fy}} = 123,69$

$$Fe = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 200000}{(45,5)^2} = 952,22$$

$$\emptyset Pn = 0,9 . \left[0.685^{\frac{fy}{fe}}\right] . fy . Ag$$

$$= 0{,}9 \cdot \left[0.685^{\frac{250}{952{,}22}}\right] \cdot 2500 \cdot 61{,}44$$

= 125168,59 Kg > Pu = 812,84 Kg

3. Kontrol Geser [SNI 1729:2015 Pasal G6]

$$\frac{h}{tw} = \frac{200}{8} < 2{,}24\sqrt{\frac{E}{fy}}$$

$$= 25 < 63{,}35 \ (Cv = 1)$$

ØVn = 0,9 x 0,6 x Fy x Aw x Cv
= 0,9 x 0,6 x 2500 x (2x20x0,8) x 1
= 43200 Kg > Vu = 540,84 Kg **(Memenuhi)**

4. Kontrol Puntir [SNI 1729:2015 Pasal H3.1]

$$\frac{h}{tw} = \frac{200}{8} < 2{,}45\sqrt{\frac{E}{fy}}$$

$$= 25 < 69{,}3$$

Maka
ØTn = 0,9 x Fcr x c
= 0,9 x (0,6xFy) x (2(B-t)(H-t)t-4,5(4- π)t^3)
= 0,9 x (0,6x2500) x (2(20-0,8)(20-0,8)0,8-4,5(4- π)$0{,}8^3$)
= 789592,97 Kg.cm
= 7895,96 Kg.m > Tu = 163,67 Kg.m **(Memenuhi)**

5. Kontrol Interaksi [SNI 1729:2015 Pasal H3.2]

Kontrol interaksi dilakukan dengan menambahkan rasio aksial, momen, geser, dan torsi dalam perhitungan sebagai beriku :

$$= \frac{Pu}{ØPn} + \frac{Mu}{ØMn} + \frac{Vu}{ØVu} + \frac{Tu}{ØTu}$$

$$= \frac{812{,}84}{125168{,}59} + \frac{617{,}26+2{,}6}{9958{,}95} + \frac{540{,}84+104{,}45}{43200} + \frac{163{,}67}{7895{,}96}$$

= 0,1044 < 1 **(Memenuhi)**

4.2.1.5 Sambungan Pipa dan Tiang Sandaran

Pada sambungan antara pipa dan tiang sandaran digunakan las bersudut (*fillet*) dengan spesifikasi sebagai beikut :

Mutu las, E70xx = 482 MPa
Tebal las minimum = 3 mm [SNI T 03 2005 Tabel 15]

Tebal las rencana, tw = 3 mm
Tebal las efektif, w = 0,707 . 3 = 2,121 mm
Panjang efektif las. Lw = π.D = π.88,9 = 279,29 mm
Gaya geser, Vu = 132,42 Kg
Gaya geser nominal
ØVn = 0,75 x w x Lw x 0,6 x Fexx
= 0,75 x 3 x 279,29 x 0,6 x 482
= 128484,83 N
= 13110,7 Kg > Vu = 132,42 Kg **(Memenuhi)**

4.2.2 Perencanaan Trotoar

Perencanaan trotoar jembatan mengikuti peraturan RSNI T-02-2005, semua trotoar harus direncanakan untuk memikul beban nominal yang berdasarkan luasannya

Beban pejalan kaki = 5 kPa = 500 kg/m2
Lebar trotoar = 1 m
Data perencanaan kerb:
h = 250 mm
γ^{u}_{TP} = 1,8
f'c = 35 MPa
fy = 400 MPa
Øtl = 13 mm (tulangan lentur)
Øts = 10 mm (tulangan susut)
Beban yang bekerja:

1. Akibat Beban Mati:
   q_D (Berat Trotoar/Kerb)
   q_D = 0,25 . 1 . 2400 . 2
   = 960 kg/m
2. Akibat Beban Hidup:
   q_l = 500 . 1 . 1,8
   = 900 kg/m

   Momen yang bekerja
   Mu = 1/8 x qu x l^2

$= 1/8 \text{ x } 1860 \text{ x } 1^2$
$= 232{,}5$ kg.m
$= 23{,}25 \text{ x } 10^5$ N.mm

3. Perhitungan tulangan Kerb:
   f`c beton = 35 MPa
   Menurut SNI 2847:2013, nilai β_1 untuk beton mutu lebih dari 30 MPa adalah:

   $\beta_1 = 0{,}85 - 8\left(\frac{f'c-30}{1000}\right)$
   $= 0{,}85 - 8\left(\frac{35-30}{1000}\right)$
   $= 0{,}81$

   fy tul. = 400 MPa
   Decking beton = 40 mm
   Ø tulangan = Ø13 mm

   d = h - (0,5. Ø tulangan) – decking
   = 200 – (0,5 x 13) – 40
   = 153,5 mm

   $\rho b = \beta \text{ x } \frac{0{,}85\,.\,f'c}{fy} \text{ x } \frac{600}{600+fy}$
   $= 0{,}81\text{x} \frac{0{,}85\,.\,35}{400} \text{ x } \frac{600}{600+400}$
   $= 0{,}0361$

   $\rho_{max} = 0{,}75 \text{ x } \rho b$
   $= 0{,}75 \text{ x } 0{,}0361$
   $= 0{,}0271$

   $\rho_{min} = \frac{1{,}4}{fy}$
   $= \frac{1{,}4}{360}$
   $= 0{,}004$

   $m = \frac{fy}{0{,}85\,.\,f'c}$
   $= \frac{400}{0{,}85\,.\,35}$
   $= 13{,}445$

Rn $= \frac{M_u}{\emptyset b d^2}$

$= \frac{23{,}25 \text{ x } 10^5}{0{,}8\ x\ 1000\ x\ 206{,}75^2}$

= 0,0702 N/mm^2

ρ $= \frac{1}{m}\left\{1 - \sqrt{1 - \frac{2.m.Rn}{fy}}\right\}$

$= \frac{1}{13{,}445}\left\{1 - \sqrt{1 - \frac{2.13{,}445.0{,}702}{400}}\right\}$

= 0,000176

$\rho_{pakai} = \rho_{min}$

= 0,004

As perlu = ρ.b.d

= 0,004 x 1000 x 206,75

= 827 mm^2

Dipasang tulangan Ø13-150 (As pakai = 884,48 mm^2)

Cek kekuatan:

Mn = As.fy.d

= 884,48 x 400 x 206,75

= 73.146.496 Nmm = 73,15 kNm

ØMn = 0,9 x 73,15

= 65,84 kNm > Mu = 2,33 kNm (OK)

Tulangan susut dipakai Ø10

d = h – decking – Øtulangan – Øsusut/2

= 250 – 40 – 13 – 10/2

= 192 mm

As perlu = 0,0018 x b x d

= 0,0018 x 1000 x 192

= 345,6 mm^2

Dipasang tulangan Ø10 – 200 (As pakai = 392,5 mm^2)

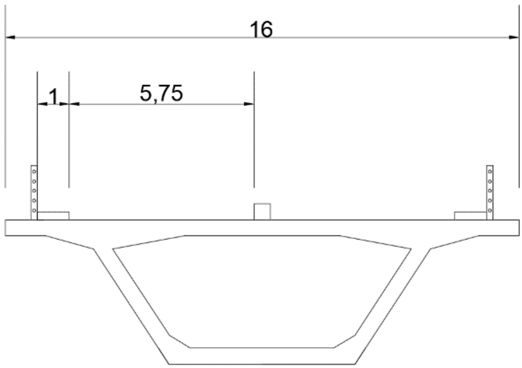
4.3 Permodelan dan Analisa Struktur

Permodelan struktur jembatan *extradosed* ini menggunaan program bantu MIDAS CIVIL. Beban yang bekerja pada jembatan ini berupa beban statik dan dinamik, serta dilakukannya staging analisis.

Beban yang termasuk beban static antara lain adalah beban mati, beban mati tambahan, beban bidup, dan beban angin. Untuk beban dinamik sendiri berupa beban gempa dengan analisa respon spectrum. Sedangkan untuk analisis staging dimodelkan seusai dengan metode pelaksanaan pendirian jembatan *extradosed*.

4.3.1 Analisa Pembebanan

Permodelan dan analisa pembebanan pada jembatab *extradosed* ini dilakukan sesuai dengan kondisi jembatan dengan lebar jalan 2 x 5,75 m seperti tertera pada gambar 4.6



Gambar 4.6 Penampang lantai kedaraan

4.3.1.1 Beban Mati (MS)

Beban mati sendiri akan diperhitungkan secara otomatis oleh program bantu MIDAS CIVIL saat dilakukan permodelan. *Software* akan memperhitungkan beban mati sesuai dengan dimensi dan spesifikasi mutu yang di *input*.

4.3.1.2 Beban Mati Tambah (MA)

Beban mati tambah berasal dari beban aspal, beban trotoar, dan beban railing jembatan.

a) Berat aspal

qu_{aspal} = tebal aspal x Bj aspal x lebar jalan
= 0,07m x 22kN/m^3 x (5,75x2)
= 8,855 kN/m

b) Berat trotoar

$qu_{trotoar}$ = tebal trotoar x Bj beton x lebar trotoar
= 0,25m x 24kN/m^3 x 2
= 12 kN/m

c) Berat railing jembatan

$qu_{railing}$ = 0,947 x 2
= 1,894 kN/m

4.3.1.3 Beban Hidup (TD)

Beban hidup sendiri terdiri dari beban terbagi rata (BTR) dan beban garis terpusat (BGT) sesuai dengan SNI 1725-2016 pasal 8.3.2.

a) Beban terbagi rata

BTR = 9 x (0,5+15/L) kN/m^2
= 9 x (0,5+15/260)
= 5,019 kN/m^2

q_{BTR} = BTR x lebar jalan
= 5,019 x (5,75x2)
= 57,721 kN/m

b) BGT = 49 kN/m

q_{BGT} = BGT x (1+FBD) x lebar jalan
= 49 x (1+30%) x (5,75x2)
= 732,55 kN

4.3.1.4 Beban Pejalan Kaki (TP)

Besar beban pejalan kaki adalah 5 kPa menurut SNI 1725-2016 pasal 8.9

qu_{TP} = TP x lebar trotoar

= 5 kN/m^2 x 2
= 10 kN/m

4.3.1.5 Beban Rem (TB)

Berdasarkan SNI 1725-2016 pasal 8.7 beban rem harus diabil yang terbesar dari :

- 25% dari berat gandar truk desain
  P_{TB1} = 0,25 x 225 kN
  = 56,25 kN
- 5% dari berat truk rencana ditambah beban BTR
  P_{TB2} = 0,05 x (500 kN + 5,019x5,75x9)
  = 37,987 kN

Maka diambil beban rem ssebesar 56,25 kN yang ditempatkan di semua lajur rencana yang bekerja secara horizontal kearah longitudinal dan setinggi 1,8 m dari lantai kendaraan.

4.3.1.6 Beban Angin (EW)

Beban angin terdiri dari 2 jenis beban yaitu, beban angin pada structural (EW_S) dan beban angin pada kendaraan (EW_I). Pada perencanaan ini arah angin diasumsikan bekerja secara tegak lurus kearah jembatan maupun kencdaraan dan dihitung berdasarkan SNI 1725-2016 pasal 9.6.1.

Untuk jembatan atau bagian jembatan dengan elevasi lebih tinggi dari 10000 mm di atas permukaan tanah atau permukaan air, kecepatan angin rencana V_{DZ} hrus dihitung dengan persamaan berikut :

$$V_{DZ} = 2{,}5\ V_0 \left(\frac{V_{10}}{V_B}\right) \ln \left(\frac{Z}{Z_0}\right)$$

dimana :

V_{DZ} = kecepatan angin rencana pada elevasi rencana (km/jam)

V_{10} = kecepatan angin pada elevasi 10000 mm diatas permukaan tanah (km/jam)

V_B = kecepatan angin rencana yaitu 90 hingga 126 km/jam pada elevasi 1000 mm (km/jam)

Z = elevasi struktur diukur dari permukaan tanah atau dari permukaan air di mana beban angin rencana dihitung (Z > 10000 mm)

V_0 = kecepatan gesekan angin, yang merupakan karakteristik meteorology, sebagaimana ditentukan dalam tabel 4.6, untuk berbagai macam tipe permukaan di hulu jembatan

Z_0 = Panjang gesekan di hulu jembatan, yang merupakan karakteristik meterologi, ditentukan pada tabel 4.8

V_{10} diperoleh dari :

- Grafik kecepatan angin dasar untuk berbagai periode ulang.
- Survei angin pada lokasi jembatan.
- Jika tidak ada data yang lebih baik, perencanaan dapat mengasumsikan bahwa $V_{10} = V_B = 90$ s/d 126 km/jam

Berikut adalah nilai V_0 dan Z_0, untuk lebih jelasnya lihat tabel 4.6.

Tabel 4.6 Nilai V_0 dan Z_0

Kondisi	Lahan terbuka	Sub urban	Kota
V_0 (km/jam)	13,2	17,6	19,3
Z_0 (mm)	70	1000	2500

Karena jembatan terletak di daerah sub urban maka :

$$V_{DZ} = 2{,}5 \times 17{,}6 \left(\frac{90}{90}\right) \ln\left(\frac{11834}{1000}\right)$$
$$= 108{,}723 \text{ km/jam}$$

a. Beban angin pada structural (EW_S)

Tekanan angin rencana (MPa) dapat ditetapkan menggunakan persamaan berikut :

$$P_D = P_B \left(\frac{V_{DZ}}{V_B}\right)^2$$

dimana :

P_B = tekanan angin dasar seperti ditentukan tabel 4.7

Tabel 4.7 Tekanan angin dasar

Komponen bangunan atas	Angin tekan (MPa)	Angin hisap (MPa)
Rangka, kolom, dan pelengkung	0,0024	0,0012
Balok	0,0024	N/A
Permukaan datar	0,0019	N/A

Gaya total beban angin tidak boleh diambil kurang dari 4,4 kN/m pada bidang tekan dan 2,2 kN/m pada bidang hisap pada struktur rangka dan pelengkup, serta tidak kurang dari 4,4 kN/m pada balok atau gelagar.

$P_D \quad = 0{,}0024 \left(\frac{108{,}723}{90}\right)^2$

$\quad\quad = 0{,}0035$ MPa

$EW_S = P_D$ x tinggi gelagar

$\quad\quad = 0{,}0035$ x (3000 ~ 4500 mm)

$\quad\quad = 10{,}507$ ~ 14,01 kN/m

Karena EW_S rencana lebih besar dari 4,4 kN/m, maka diambil EW_S rencana sebesar 10,507 ~ 14,01 kN/m.

b. Beban angin pada kendaraan (EW_I)

Berdasarkan SNI 1725-2016 pasal 9.6.1.2 tekanan angin rencana harus memikul gaya akibat tekanan angin pada kendaraan, dimana tekanan tersebut harus di asumsikan sebagai tekanan menerus sebesar1,46 kN/m. tegak lurus dan bekerja 1,8 m di atas permukaan jalan

$EW_I = 1{,}46$ kN/m

4.3.1.7 Beban Temperatur (EU_n)

Besaran rentang simpangan akibat beban temperature ∆T harus berdasarkan temperature maksium dan minimum yang didefinisikan dalam desain sebagai berikut menurut SNI 1725-2016 pasal 9.3.1.1

$$\Delta T = \alpha L \; (Tmax - Tmin)$$

dimana :

L = panjang komponen jembatan (mm)

α = koefisien muai temperature (mm/mm/^{o}C)

Berikut adalah nilai koefisien muai temperatur, untuk lebih jelasnya lihat tabel 4.8 dan 4.9.

Tabel 4.8 Sifat bahan akibat temperatur

Bahan	Nilai koefisien α	Modulus Elastisitas (MPa)
Baja	$12x10^{-6}$	200.000
Beton - kuat tekan < 30 MPa - kuat tekan > 30 MPa	 $10x10^{-6}$ $11x10^{-6}$	$4700\sqrt{f'c}$

Tabel 4.9 Temperatur rata-rata jembatan

Tipe bangunan atas	Tmin	Tmax
Lantai beton di atas gelagar/box beton	15 ^{o}C	40 ^{o}C
Lantai beton diatas gelagar, box, atau ranka baja	15 ^{o}C	40 ^{o}C
Lantai pelat baja di atas gelagar, box, atau rangka baja	15 ^{o}C	45 ^{o}C

$\Delta T = 11x10^{-6}$ x 260000 (40 – 15)

= 71,5 mm

4.3.1.8 Beban Gempa

Untuk pembebanan gempa pada desain jembatan ini digunakan metode respons spectrum dengan program bantu MIDAS CIVIL dan peta gempa.pusjatan.pu.go.id. Berdasarkan SNI 2833-2016 pasal 5.3.1, untuk kasifikasi jenis tanah dapat diilihat pada tabel 4.10.

Tabel 4.10 Kelas situs

Kelas Situs	$\bar{N}$
C. Tanah Sangat Padat dan Batuan Lunak	$\bar{N} > 50$
D. Tanah Sedang	$15 < \bar{N} < 50$
E. Tanah Lunak	$\bar{N} < 15$

$$\bar{N} = \frac{\sum_{i=0}^{n} t_i}{\sum_{i=0}^{n} \left(\frac{t_i}{\bar{N}_i}\right)}$$

dimana :

t_i = tebal lapisan tanah ke-i

N_i = nilai hasil uji penetrasi standart lapisan ke-i

Didapatkan hasil drill log tanah di kecamatan widang kabupaten tuban per 2 meteran. Unutk lebih jelasnya dapat melihat tabel 4.11 dan 4.12.

Tabel 4.11 Pengolaan data tanah 1

Kedalaman	Tebal (t_i)	Nilai SPT	$\frac{t_i}{\bar{N}_i}$
2	2	10	0,2
4	2	4	0,5
6	2	13	0,154
8	2	22	0,091
10	2	36	0,056
12	2	32	0,063
14	2	43	0,047
16	2	24	0,083
18	2	43	0,047
20	2	50	0,04
22	2	50	0,04
24	2	50	0,04
26	2	10	0,2

28	2	45	0,044
30	2	49	0,041
Total	30		1,644

$$\overline{\mathrm{N}} = \frac{30}{1{,}644} = 18{,}248$$

Tabel 4.12 Pengolaan data tanah 2

Kedalaman	Tebal (t_i)	Nilai SPT	$\frac{t_i}{\overline{N}_i}$
2	2	12	0,1667
4	2	10	0,2
6	2	13	0,154
8	2	30	0,067
10	2	25	0,080
12	2	30	0,067
14	2	25	0,080
16	2	37	0,054
18	2	50	0,040
20	2	47	0,0426
22	2	50	0,04
24	2	50	0,04
26	2	50	0,04
28	2	40	0,050
30	2	44	0,045
Total	30		1,166

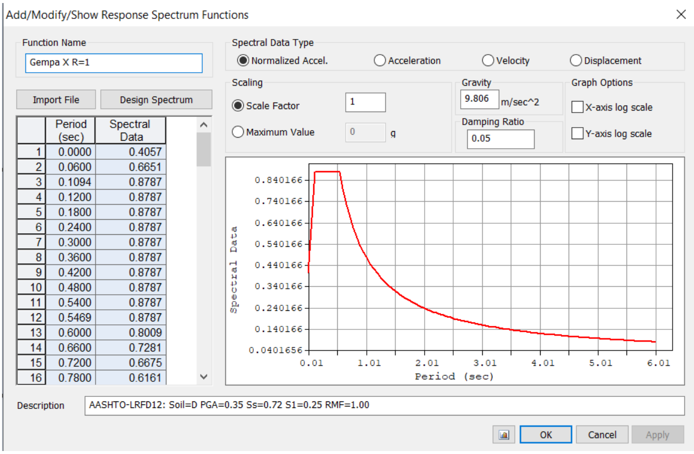
$$\overline{\mathrm{N}} = \frac{30}{1{,}166} = 25{,}73$$

Berdasarkan data diatas tanah dikategorikan tanah sedang (SD) dengan, 15 < $\overline{N}$ < 50. Dengan jenis tanah SD didapatkan data-data berikut melalui peta gempa 1000 tahun dengan kemungkinan terlampaui 7% dalam 75 tahun memiliki angka.

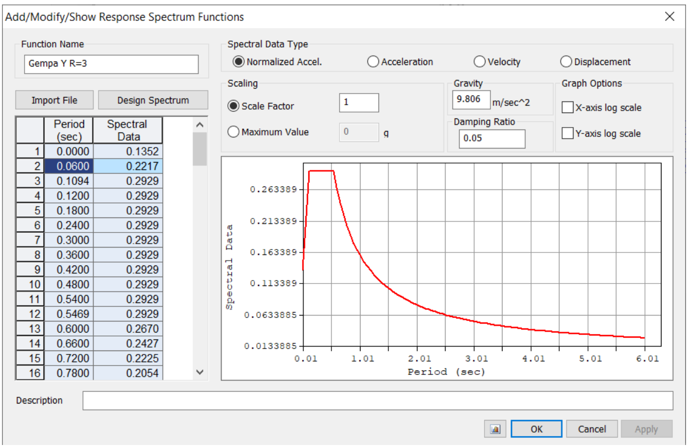
- Percepatan pucak (PGA) = 0,354

- Percepatan 0,2 detik (S_S) = 0,716
- Percepatan 1 detik (S_1) = 0,254

Dari data diatas dilakukan input data respon spektra, berupa PGA, S_S, S_1, dan factor modifikasi respon R untuk gempa x sebesar R = 1 dan gempa y R = 3. Untuk hasil respon spectra desain dapat dilihat pada gambar 4.7 dan gambar 4.8



Gambar 4.7 Diagram respon spektra gempa x



Gambar 4.8 Diagram respon spektra gempa y

Berdasarkan SNI 2833-2016 pasal 5.3.2 nilai Fa dan Fv dengan kelas situs tanah sedang (SD).

- F_{PGA} = 1,146
- Fa = 1,2272
- Fv = 1,892

Dengan perumusan respon spektra sebagai berikut

$A_S = F_{PGA} \times PGA$

$= 1,146 \times 0,354$

$= 0,4057$

$S_{DS} = Fa \times S_S$

$= 1,2272 \times 0,716$

$= 0,8787$

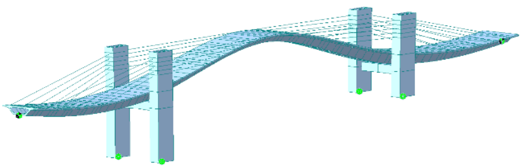
$S_{D1} = Fv \times S_1$

$= 1,892 \times 0,254$

$= 0,4805$

➢ Periode struktur

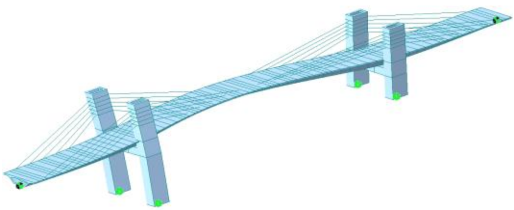
Berikut adalah gambe mode 1 yang menunjukan lentur X jembatan yang bisa dilihat pada gambar 4.9.



Gambar 4.9 Vibration Mode Shape Pada Mode 1

Tx (MIDAS Mode 1) = 1,013153

Sedangkan untuk lentur Y jembatan dilihat pada gambar 4.10.



Gambar 4.10 Vibration Mode Shape Pada Mode 2

Ty (MIDAS Mode 2) = 0,570685

$T_s = \frac{S_{D1}}{S_{Ds}}$

$= 0,5468$

$T_0 = 0,2 \times T_s$

$= 0,1094$

- Kofisien respon gempa elastic

  Dikarenakan periode lebih besar dari T_s, maka menggunakan rumusan sebagai berikut :

$C_{smx} = \frac{S_{D1}}{Tx}$ $C_{smy} = \frac{S_{D1}}{Ty}$

$= 0,47426$ $= 0,842$

- Perhitungan berat mati jembatan

  Berikut adalah nilai dari berat jembatan pada table 4.13.

Tabel 4.13 Beban Mati Jembatan

Jenis		Luas (m^2)	Panjang (m)	Volume (m^3)	BJ (kN/m^3)	Berat (kN)
Girder	4,5	16,897	8	135,17	24	3244,147
	Tapered	16,338	252	4117,17	24	98812,03
Tower		20	136	2720	24	65280
Balok		6	36	216	24	5184
Kabel	1	0,01442	201,79	2,910	77,01	224,09
Kabel	2	0,01554	266,83	4,147	77,01	319,33
Kabel	3	0,01554	331,90	5,158	77,01	397,20
Kabel	4	0,01666	396,99	6,614	77,01	509,33
Kabel	5	0,01834	462,08	8,475	77,01	652,63
Total						174622,8

Perbandingan berat manual dan MIDAS

Berat manual = 174622,8 kN

Berat MIDAS = 175099,2 kN

Selisih = 0,273 %

- Menghitung gaya gempa horizontal

Gaya gempa statis

$E_{QX} = \frac{C_{smx}}{R} \times W_t$

$= \frac{0{,}47426}{1} \times 175099{,}2$

$= 83042{,}546$ kN

$E_{QY} = \frac{C_{smY}}{R} \times W_t$

$= \frac{0{,}842}{3} \times 175099{,}2$

$= 49144{,}508$ kN

Gaya gempa dinamis dari MIDAS

E_{QX} = 111534,4 kN

E_{QY} = 37494,7 kN

➢ Kontrol gaya gempa dinamis

Berdasarkan SNI 1726-2012 pasal 7.9.4.1 beban gempa dinamis harus dikontrol terhadap beban gempa static.

$V_{dinamis} > 0{,}85 \times V_{statis}$

- Arah X

  $V_{dinamisX} > 0{,}85 \times V_{statisX}$

  $111534{,}4 > 0{,}85 \times 83042{,}546$

  $111534{,}4 > 70586{,}164$ kN **(OK)**

- Arah Y

  $V_{dinamisY} > 0{,}85 \times V_{statisY}$

  $37494{,}7 > 0{,}85 \times 49144{,}508$

  $37494{,}7 > 41772{,}832$ kN **(Perlu Kalibrasi)**

Dilakukan pembesaran gaya gempa Y sebesar

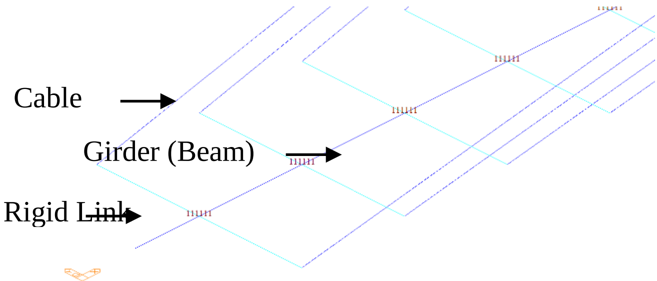
$\frac{0{,}85 \times V_{statisY}}{V_{dinamisY}} = \frac{41772{,}832}{37494{,}7}$

$= 1{,}115$

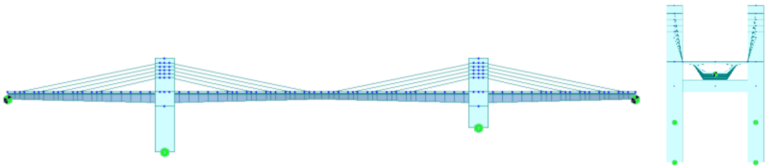
$V_{dinamisY} = 41806{,}4 \text{ kN} > 41772{,}832$ kN **(OK)**

4.3.2 Permodelan Jembatan

Permodelan dilakukan dengan program bantu MIDAS CIVIL dalam 3D model. Dimana elemen kabel dimodelkan sebagai elemen *truss* yang dihubungkan dengan *rigid link support*, sedangkan *main girder* dimodelkan sebagai elemen *beam* dengan *fishbone* model (*one frame*). Untuk lebih jelasnya dapat melihat gambar 4.11 dan gambar 4.12



Gambar 4.11 Permodelan elemen jembatan



Gambar 4.12 Tampak memanjang dan melintang model

4.3.3 Analisa Struktur

Berdasarkan Analisa dinamis struktur menggunakan program bantu MIDAS CIVIL didapatkan hasil tiap mode yang ditabelkan dibawah ini. Untuk lebih jelasnya lihat tabel 4.14.

Tabel 4.14 Periode struktur tiap vibration mode shape

Mode No	Period (sec)	Mode No	Period (sec)
1	1,013153	11	0,225806
2	0,570685	12	0,22469
3	0,489715	13	0,214151

4	0,42521	14	0,183292
5	0,413188	15	0,17964
6	0,39615	16	0,176244
7	0,371999	17	0,158113
8	0,285819	18	0,151664
9	0,24422	19	0,143154
10	0,229932	20	0,136423

Dan berikut adalah nilai partisipasi massa yang nilainya harus lebih besar dari 90%. Untuk lebih jelasnya dapat dilhat pada tabel 4.15.

Tabel 4.15 Modal Praticipation masses

Mode No	TRAN-X		TRAN-Y	
	MASS(%)	SUM(%)	MASS(%)	SUM(%)
1	0,09	0,09	0	0
2	0	0,09	55,48	55,48
3	18,87	18,96	0	55,48
4	69,62	88,58	0	55,48
5	0,71	89,28	0	55,48
6	0	89,28	0,01	55,49
7	0	89,29	0	55,49
8	0	89,29	12,29	67,78
9	0	89,29	0	67,78
10	0	89,29	1,42	69,2
11	0	89,29	0	69,2
12	0	89,29	23,22	92,42
13	0	89,29	0	92,42
14	0	89,29	0,01	92,43
15	0,13	89,42	0	92,43
16	0	89,42	0,07	92,5
17	0	89,42	0	92,5

18	0,04	89,46	0	92,5
19	1,22	90,68	0	92,5
20	0	90,68	0,03	92,53

Dapat dilihat pada tabel diatas bahwa nilai partisipasi massa sebesar 90% pada sumubu X didapatkan pada mode ke 19 sedangkan pada sumbu Y didapatkan pada mode ke 12.

4.3.4 Staging Analisis

Metode pelaksanaan (*Staging Analysis*) konstruksi jembatan *extradosed* ini didesain dengan *cantilever method*. Urutan pekerjaannya dimulai dengan pekerjaan *pylon* dari beton bertulang dan pemasangan box girder pertama pada balok tumpuan. Dilanjutkan dengan pemasangan *traveler* pada bagian atas box girder pertama. Dilanjutkan dengan pemasangan box girder kedua dengan bantuan *form treveler*, pemasangan *ducting* kabel pada box girder, pemasangan kabel dan dilanjutkan *jacking* kabel. Begitu seterusnya hingga semua box terpasang.

Sedangkan untuk metode analisisnya dilakukan dengan metode *demolishing procedure* melalui *backward solution*. Dimulai dari keadaan final jembatan dilanjutkan dengan melepas bagian per bagian jembatan hingga pada keadaan awal. Semua tahapan tersebut akan dibantukan dengan program bantu MIDAS CIVIL sehingga didapatkan hasil gaya per tahapan analisa.

4.3.4.1 Pembebanan Staging Analisis

Spesifikasi *form treveler* yang akan digunakan dalam *staging analysis* ini mengikuti spesifikasi *form treveler overhead model* dari Handan China Railway Bridge Machinery Co.Ltd. Untuk lebih jelasnya dapat melihat gambar 4.13.

Gambar 4.13 Form traveler milik Handan China Railway Bridge Machinery Co.Ltd

Berikut adalah spesifikasi dari *form traveler* dapat dilihat pada tabel 4.16.

Tabel 4.16 FT-S Series Form-Traveler (Over Head Model)

Item	Description		Specification
1	Model		FT-S
2	Capacity		100t ~ 480t
3	Segment Length		3.5m ~ 7.0m
4	Deck Width		5m ~ 35m
5	Bridge Curvature Radius		100m-unlimited
6	Bridge Type		Balance Cantilever Box Girder or Cable Stay
7	Launching Mechanism		Hydraulic
8	Formwork Material		Metal Sheet or Plywood Sheet
9	Shape of Bridge Section		Any shape
10	Production Cycle time		5 days – 7 days depend on site condition, concreting capacity, concrete design, pier height, reinforcement fabrication method etc.
11	Max. Bridge Slope	Longitudinal	7%
		Transverse	5%

Pada saat pelaksanaan *staging analysis* beban box girder yang akan diangkat dipikul oleh *form traveller* yang kemudian akan disalurkan pada lantai kendaraan jembatan, untuk konfigurasi pembebanan dapat dilihat pada tabel 4.17.

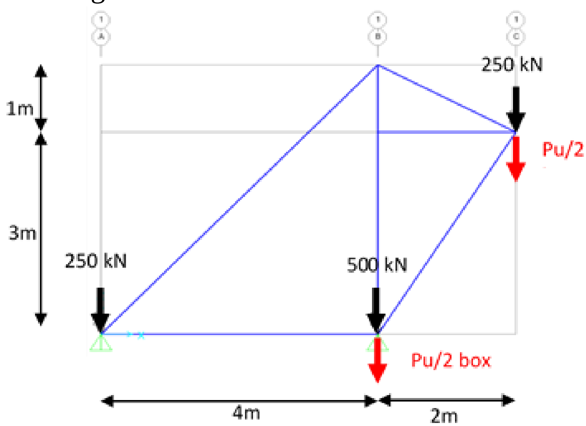
Tabel 4.17 Konfigurasi beban saat staging

Kasus	Konfigurasi Beban
1	MS + *Form Traveller*

Berikut contoh perhitungan beban yang akan dipikul lantai kendaraan pada saat tahap konstruksi pengangkatan segmen 1.

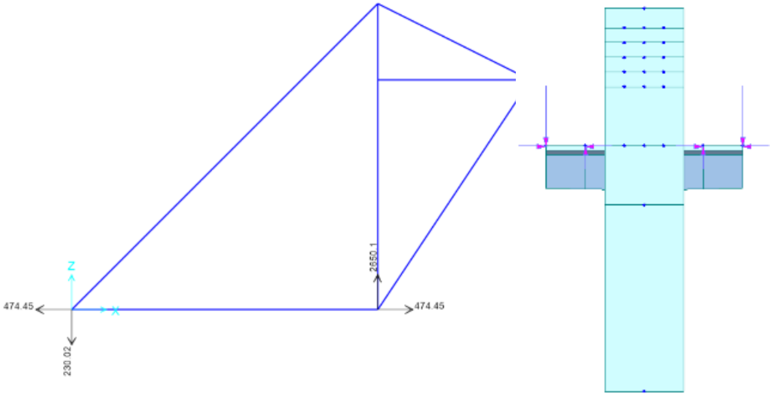
- Berat sendiri box girder segmen 1 (4m)
  Pu = A x BJ beton x λ
  = (14,81+14,77)/2 x 24 x 4
  = 1420,077 kN (dipikul pada 2 titik)
- Berat *form traveller* tipe *overhead trianle* berdasakan produk Handan China Railway Bridge Machinery Co.Ltd
  Pu = 1000 kN

Berikut adalah contoh permodelan pembebanan *form traveler* yang sebelumnya dilakukan analisa dengan program bantu SAP 2000 V20.2 dengan mengabaikan beban profil baja yang digantikan beban terpusat sebesar 1000 kN. Unutk lebih jelasnya dapat melihat gambar 4.14.



Gambar 4.14 Permodelan Form Traveler Pada Sap

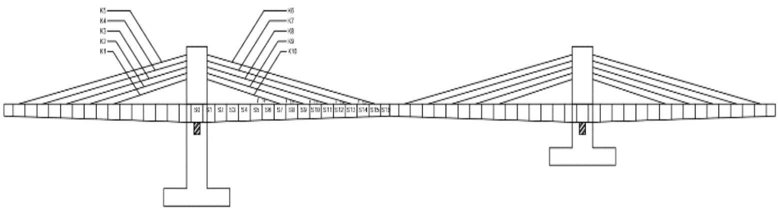
Setelah dilakukan permodelan *form treveler* pada SAP 2000 V20.2 dan didapatkan reaksi pada perletakan joint. Dilakukanlah input beban hasil dari reaksi tersebut dengan merubah arah bebannya sebagai bentuk aksi beban pada lantai kendaraan. Untuk lebih jelasnya dapat melihat gambar 4.15.



Gambar 4.15 Hasil reaksi SAP 2000 dan aksi MIDAS CIVIL

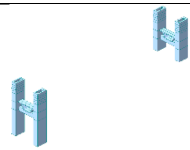
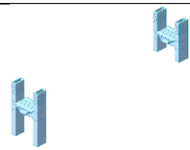
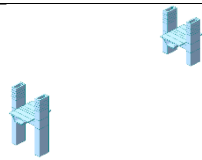
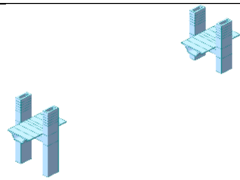
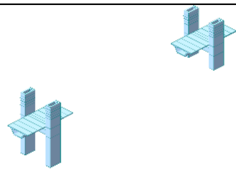
4.3.4.2 Tahapan Analisa Staging

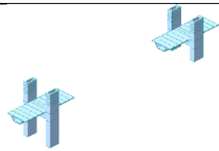
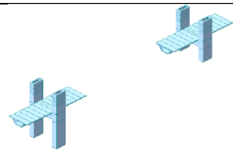
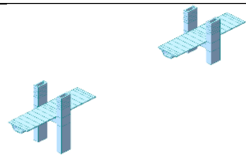
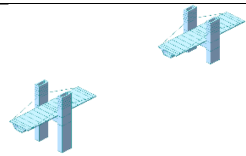
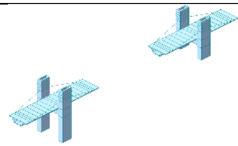
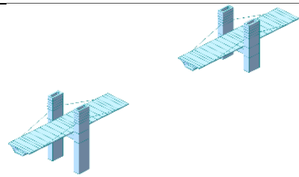
Berikut adalah urutan pelaksanaan *staging analysis* dengan program bantu MIDAS CIVIL. Dimana urutan pelaksanaan dilapangan sama dengan urutan analisisnya, namun pelaksanaan dilapangan merupakan *forward method* sedangkan analisis MIDAS CIVIL menggunakan *backward solution*. Sebelumnya dilakukan penomoran terhadap box girder dan kabel. Untuk lebih jelasnya dapat melihat gambar 4.16 dan tabel 4.18.

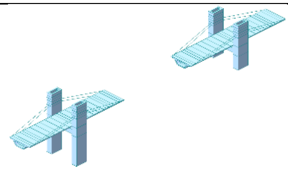
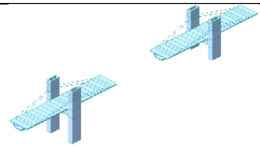
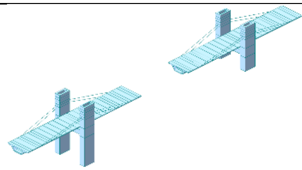
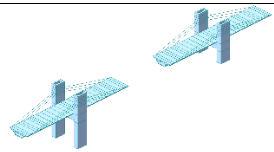
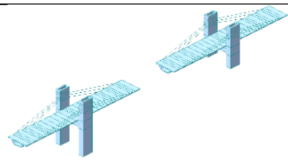
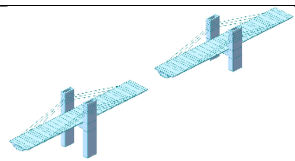


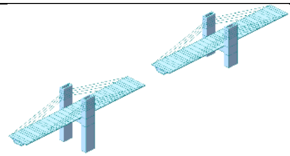
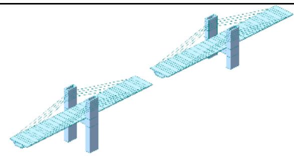
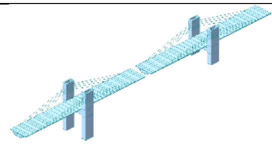
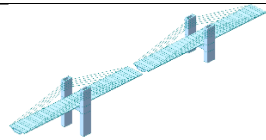
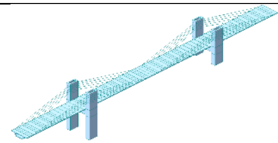
Gambar 4.16 Penomoran Box Dan Kabel

Tabel 4.18 Permodelan Staging

Stage	Penjelasan	Beban Angkut (kN)	Model
21	Pembangunan *pylon* dan box segmen 0		
20	Pemasangan box segmen 1 dan *stressing*	1420,08	
19	Pemasangan box segmen 2 dan *stressing*	1416,8	
18	Pemasangan box segmen 3 dan *stressing*	1413,52	
17	Pemasangan box segmen 4 dan *stressing*	1410,24	

16	Pemasangan box segmen 5 dan *stressing*	1406,95	
15	Pemasangan box segmen 6 dan *stressing*	1403,67	
14	Pemasangan box segmen 7 dan *stressing*	1400,39	
13	Pemasangan kabel 1 dan 10		
12	Pemasangan box segmen 8 dan *stressing*	1397,11	
11	Pemasangan box segmen 9 dan *stressing*	1393,83	

10	Pemasangan kabel 2 dan 9		
9	Pemasangan box segmen 10 dan *stressing*	1390,55	
8	Pemasangan box segmen 11 dan *stressing*	1387,27	
7	Pemasangan kabel 3 dan 8		
6	Pemasangan box segmen 12 dan *stressing*	1383,99	
5	Pemasangan box segmen 13 dan *stressing*	1380,71	

4	Pemasangan kabel 4 dan 7		
3	Pemasangan box segmen 14 dan *stressing*	1377,43	
2	Pemasangan box segmen 15 dan *stressing*	1374,15	
1	Pemasangan kabel 5 dan 6		
0	Pemasangan closure	1028,15	

4.3.4.3 Hasil Analisa Staging

Berikut adalah output gaya dalam dari outpus MIDAS CIVIL, untuk lebih jelasnya dapat melihat tabel 4.19 hingga 4.22.

Tabel 4.19 Output Gaya Dalam Box Girder Saat Staging

Elmn	SG	Shear-z (kN)	Moment-y (kN,m)	Elmn	SG	Shear-z (kN)	Moment-y (kN,m)
165	1	-19923,5	-578164	50	12	15136.81	-283111
165	2	-19266,5	-519620	50	13	13551.24	-223880

165	3	-19745,3	-550106	17	14	11748.41	-171900
165	4	-18876,2	-483270	17	15	12800.89	-197795
50	5	17817,7	-421103	17	16	11000.21	-145505
50	6	18530,88	-458035	17	17	9416.21	-105096
50	7	17435,32	-392785	17	18	7824.5	-70889.9
50	8	16175,08	-335239	17	19	6224.96	-42946.5
50	9	17014,7	-369773	17	20	4617.47	-21327.7
50	10	15676,15	-306681	164	21	-3231.94	-5652.84
50	11	14177,45	-252040				

Tabel 4.20 Output gaya dalam pylon saat staging

SG	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (Kn.m)	Moment-y (kN.m)	Moment-z (kN.m)
1	-47109	8321,21	0	-1662,21	965,41	-39093,7
1	-37843	11635,9	0	1759,62	-965,41	120682,9
0	-9295,31	-1361,47	4316,72	-521,18	-8354,33	2285,56
0	-36902,4	11433,33	-435,08	20860,9	71660,6	118142
0	-46258,1	8176,13	435,08	-18061,8	-74091,1	-38506,2
1	-37842,9	-11635,9	0	-1759,62	-965,41	-120683

Tabel 4.21 Output gaya dalam balok saat staging

SG	Axial (kN)	Shear-y (kN)	Shear-z (kN)	Torsion (kN.m)	Momeny (kN.m)	Moment-z (kN.m)
1	-13287,2	-342,75	-21480,3	-1661,9	-97890,3	-1627,58
0	-12781,2	4263,74	20370,84	5846,2	111196	22340,46
0	-9537,59	3881,64	-21844,7	5669,6	-98217,1	17540,59
0	-12781,2	-4263,74	-21810,8	-5846,2	-99712,5	-20296,9
0	-9537,59	3881,64	-20404,7	5669,6	113030,1	-21275,8
0	-12781,2	-4263,74	-20370,8	-5846,2	111196	22340,46

Tabel 4.22 Output gaya dalam kabel saat staging

Elemen	Stage	Force (kN)
Tendon 1	SG1	4526,924
Tendon 2	SG1	4325,157
Tendon 3	SG1	3502,118
Tendon 4	SG1	2226,291
Tendon 5	SG1	698,0275
Tendon 6	SG1	2237,382
Tendon 7	SG1	2379,037
Tendon 8	SG3	2936,13
Tendon 9	SG3	3629,872
Tendon 10	SG3	3909,102

4.4 Perencanaan Box Girder

4.4.1 Perencanaan Construction Tendon

Direncanakan menggunakan tendon / kabel jenis strand *seven wires stress relieved* (7 kawat untaian). Dengan mengacu pada tabel VSL tipe ASTM A 416-06 Grade 270, berikut adalah jenis dan karakteristik tendon yang digunakan :

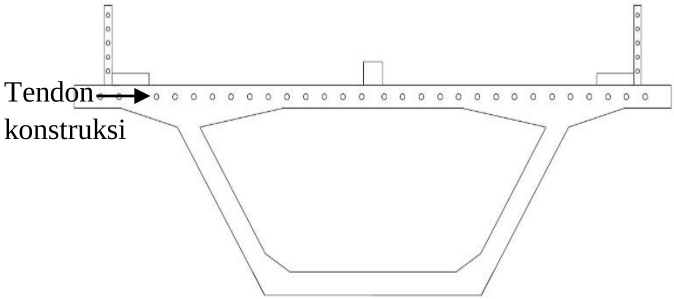
- Diameter : 15,24 mm
- Luas nominal (Ap) : 140 mm^2
- Kuat nominal tarik : 1860 MPa
- Min. breaking load : 260,7 kN
- Modulus elastisitas : 200 GPa

Unit tendon yang digunakan sebagai *construction tendon* berupa 6-31-31 dengan spesifikasi sebagai berikut :

- Steal area : 4340 mm^2
- Min. breaking load : 8082 kN

F_{ijin} = 0,9 x min. breaking load
= 0,9 x 8082
= 7273,8 kN
fs = F_{ijin}/Ap
= 7273800/4340
=1675,99 Mpa

Direncanakan letak tendon saat konstruksi jembatan diletakan secara internal pada flens atas box girder secara mendatar, unutk menghindari kesulitan pemasangan tendon saat konstruksi. untuk lebih jelasnya dapat melihat gambar 4.17.



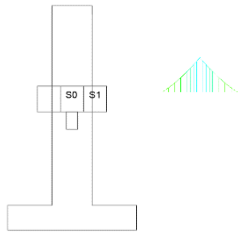
Gambar 4.17 Letak tendon konstruksi

Dengan mengambil nilai e pada joint sebagai berikut :

e = ya – 0,25
$\sigma_{ijin\ tekan}$ = 0,6 x f'c
= 0,6 x 70 MPa
= 42 Mpa
$\sigma_{ijin\ tarik}$ = 0,25 x $\sqrt{f'c}$
= 2,091 Mpa

Berikut adalah contoh perhitungan kontrol penampang saat stage 20 dan stage 1

1. <u>Stage 20</u>



Gambar 4.18 Diagram momen dan aksial SG20

Perhitungan pada segmen 1 saat stage 20 seperti gambar 4.18 dengan jumlah tendon 2.

$$\sigma_{atas} = \frac{F}{A} + \frac{F.e.ya}{Ix} + \frac{My.ya}{Ix}$$

$$= \frac{2.7273,8}{16,897} + \frac{2.7273,8x1,287x1,537}{44,4402} + \frac{-12903,77x1,537}{44,4402}$$

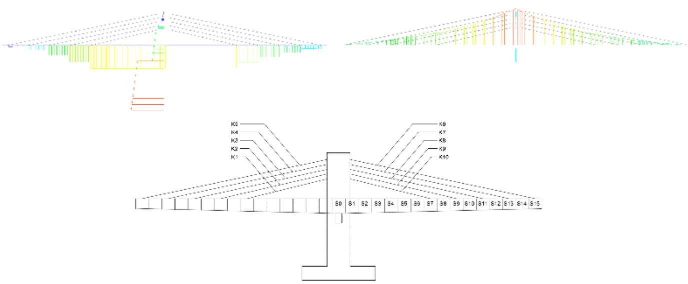
= 1,064 MPa > -2,091 MPa **(memenuhi)**

$$\sigma_{bawah} = \frac{F}{A} - \frac{F.e.yb}{Ix} - \frac{My.yb}{Ix}$$

$$= \frac{2.7273,8}{16,897} - \frac{2.7273,8x1,287x2,963}{44,4402} - \frac{-12903,77x2,963}{44,4402}$$

= 0,475 MPa < 42 MPa **(memenuhi)**

2. <u>Stage 1</u>



Gambar 4.19 Diagram momen dan aksial SG1

Perhitungan pada segmen 1 saat stage 1 seperti gambar 4.19 dengan jumlah tendon 30.

σ_{atas} $= \frac{F}{A} + \frac{F.e.ya}{Ix} + \frac{My.ya}{Ix}$

$= \frac{30.7273,8+29923}{16,897} + \frac{30.7273,8x1,287x1,537}{44,4402} + \frac{-538999x1,537}{44,4402}$

= 5,756 MPa > -2,091 MPa **(memenuhi)**

σ_{bawah} $= \frac{F}{A} - \frac{F.e.yb}{Ix} - \frac{My.yb}{Ix}$

$= \frac{30.7273,8+29923}{16,897} - \frac{30.7273,8x1,287x2,963}{44,4402} - \frac{-538999x2,963}{44,4402}$

= 31,907 MPa < 42 MPa **(memenuhi)**

Perhitungan pada segmen 15 saat stage 1 seperti gambar 4.23 dengan jumlah tendon 2.

σ_{atas} $= \frac{F}{A} + \frac{F.e.ya}{Ix} + \frac{My.ya}{Ix}$

$= \frac{2.7273,8+1767,15}{15,875} + \frac{2.7273,8x0,844x1,094}{18,915} + \frac{-9626,62x1,094}{18,915}$

= 1,181 MPa > -2,091 MPa **(memenuhi)**

σ_{bawah} $= \frac{F}{A} - \frac{F.e.yb}{Ix} - \frac{My.yb}{Ix}$

$= \frac{2.7273,8+1767,15}{15,875} - \frac{2.7273,8x0,844x2,072}{18,915} - \frac{-9626,62x2,072}{18,915}$

= 0,736 MPa < 42 MPa **(memenuhi)**

Berikut adalah tabel perhitungan kontrol penampang saat tahap stage 1 yang dapat dilihat pada tabel 4.23.

Tabel 4.23 Kontrol Penampang Box Girder

Box	n	Momen (kN.m)	Axial (kN)	σ atas (MPa)	σ bawah (MPa)	CEK
S0	30	-571612	-29080,1	4,578	34,032	OK
S1	30	-538999	-29923	5,7562	31,907	OK
S2	28	-466815	-29907,5	6,3448	28,361	OK
S3	26	-401075	-29892,4	6,8151	25,038	OK
S4	24	-341747	-29877,8	7,1567	21,961	OK
S5	22	-288801	-29863,5	7,3577	19,145	OK
S6	20	-242205	-29849,6	7,4049	16,618	OK

S7	18	-201929	-29836,3	7,2830	14,403	OK
S8	16	-175430	-21048,8	6,1166	12,622	OK
S9	14	-139447	-21036	5,9598	10,441	OK
S10	12	-116533	-12612,6	4,7663	8,721	OK
S11	10	-85457,4	-12600,2	4,5358	6,6373	OK
S12	8	-65784,3	-5765,5	3,3966	5,037	OK
S13	6	-41133,7	-5753,57	2,9953	3,230	OK
S14	4	-25716,1	-1400	1,8970	1,9340	OK
S15	2	-9626,62	-1767,15	1,1814	0,7365	OK

4.4.1.1 Kehilangan Prategang

1. Akibat perpendekan elastis beton Δes

   Pada kehilangan ini memperhitungkan pengaruh penarikan yang berturut-turut. Tetapi akan cukup teliti bila menentukan kehilangan prategang dari kabel pertama dan mengambil setengah dari nilai itu untuk kehilangan prategang rata-rata seluruh kabel. Berikut adalah persamaan yang digunakan untuk kontrol segmen 1 saat stage 1.

   $$\Delta es = K_{ES} \cdot \frac{E_s}{E_c} \cdot f_{cir}$$
   $$= 0{,}5 \text{ x } \frac{195000}{39323{,}021} \text{ x } 4{,}82$$
   $$= 11{,}95 \text{ MPa}$$

   dimana :

   f_{cir} = tegangan beton akibat gaya prategang pada tendon
   E_s = modulus elastisitas tendon prategang
   E_c = modulus elastisitas beton
   K_{ES} = 1,0 untuk struktur pratarik dan 0,5 untuk pasca tarik

2. Akibat gesekan $\Delta fs1$

   Kehilangan akibat gesekan dipengaruhi 2 hal yaitu panjang dan kelengkungan. Berikut adalah persamaan yang digunakan.

   $$\Delta fs1 = (-K.L - \mu.\alpha).fs$$
   $$= (0{,}0045 \text{ x } 4 - 0{,}2 \text{ x } 0).1675{,}99$$

= 30,168 MPa

dimana :

K = koefisien Wobble (diambil 0,0045 berdasarkan tabel 3.1)

L = Panjang titik tinjau

μ = koefisien kelengkungan (diambil 0,2)

α = sudut kelengkungan = 0^o

3. Akibat slip angkur Δfs2

Pada system pasca tarik, tendon yang ditarik kemudian dilepas dan gaya dialihkan ke angkur mengakibatkan angkur cenderung berdeformasi, sehingga tendon dapat tergelincir sedikit sebelum dijepit dengan kokoh. Besar gelincir rata-rata sekitar 2,5 mm. Berikut adalah persamaan yang digunakan.

$$\Delta\alpha = \frac{fs\,.\mathrm{L}}{\mathrm{Es}}$$

$$= \frac{1675{,}99\,.12000}{195000}$$

$$= 103{,}138 \text{ mm}$$

dimana :

Δα = deformasi pengangkuran (mm)

fs = tegangan tendon (MPa)

L = Panjang total tendon (mm)

Es = modulus elastisitas tendon (MPa)

$$\Delta fs2 = \frac{2{,}5}{\Delta\alpha}\,.\,fs$$

$$= \frac{2{,}5}{103{,}138} \times 1675{,}99$$

$$= 40{,}625 \text{ Mpa}$$

a) Tegangan efektif pada tendon

fse = *fs* - Δes - Δfs1 - Δfs2

= 1675,99 – 11,95 – 30,168 – 40,625

= 1593,247 Mpa

b) Gaya prategang efektif pada tendon

Fse = jumlah tendon x fse x Ap

= 30 x 1593,247 x 4340

= 207440,85 kN

c) Kontrol gaya prategang efektif

Contoh perhitungan pada segmen 1 saat stage 1

$$\sigma_{atas} = \frac{F}{A} + \frac{F.e.ya}{Ix} + \frac{My.ya}{Ix}$$

$$= \frac{207440{,}8+29923}{16{,}897} + \frac{207440{,}8x1{,}287x1{,}537}{44{,}4402} + \frac{-538999x1{,}537}{44{,}4402}$$

= 4,639 MPa > -2,091 MPa **(memenuhi)**

$$\sigma_{bawah} = \frac{F}{A} - \frac{F.e.yb}{Ix} - \frac{My.yb}{Ix}$$

$$= \frac{207440{,}8+33362}{16{,}897} - \frac{207440{,}8x1{,}287x2{,}963}{44{,}4402} - \frac{-505731x2{,}963}{44{,}4402}$$

= 32,194 MPa < 42 MPa **(memenuhi)**

Berikut adalah tabel perhitungan kehilangan prategang saat tahap stage 1 yang dapat dilihat pada tabel 4.24.

Tabel 4.24 Kontrol Kehilangan Prategang

Box	n	Δes (MPa)	Δfs1 (MPa)	Δfs2 (MPa)	Fse (kN)	σ atas (MPa)	σ bawah (MPa)	CEK
S0	30	9,51	30,17	40,63	207759,13	3,495	34,310	OK
S1	30	11,95	30,17	40,63	207440,85	4,639	32,194	OK
S2	28	13,12	30,17	24,38	195444,17	5,489	28,580	OK
S3	26	14,03	30,17	17,41	182166,76	6,089	25,226	OK
S4	24	14,67	30,17	13,54	168490,54	6,518	22,125	OK
S5	22	15,01	30,17	11,08	154652,17	6,791	19,292	OK
S6	20	15,03	30,17	9,38	140738,96	6,903	16,747	OK
S7	18	14,71	30,17	8,13	126788,05	6,842	14,518	OK
S8	16	12,28	30,17	7,17	112935,12	5,748	12,718	OK
S9	14	11,90	30,17	6,41	98887,42	5,643	10,523	OK
S10	12	9,46	30,17	5,80	84919,60	4,510	8,788	OK
S11	10	8,94	30,17	5,30	70810,62	4,326	6,692	OK
S12	8	6,65	30,17	4,88	56742,78	3,238	5,079	OK
S13	6	5,82	30,17	4,51	42588,07	2,879	3,261	OK

S14	4	3,66	30,17	4,20	28435,01	1,824	1,953	OK
S15	2	2,26	30,17	3,93	14232,00	1,146	0,746	OK

4.4.1.2 Kontrol Momen Retak

Momen yang menghasilkan retak-retak pertama pada balok beton prategang dihitung dengan teori elastik, dengan menganggap bahwa retak mulai terjadi saat tegangan tarik pada serat terluar beton mencapai modulus keruntuhannya. Modulus keruntuhan merupakan ukuran permulaan retak-retak yang sering kali todak terliihat oleh mata telanjang. Saat kondisi beton telah mengalami retak akibat beban berlebihan, susut atau sebab-sebab lainnya, maka retak-retak dapat terlihat pada tegangan tarik yang terkecil (Lin dan Burns, 1982). Dengan menggunakan analisa elastik beton prategang, perumusan momen retak adalah sebagai berikut :

$$M_{cr} = F\left(e + \frac{r^2}{y}\right) + \frac{fr.Ix}{y}$$

dimana :

M_{cr} = momen retak (kN.m)
F = gaya *stressing* tendon (kN)
e = jarak C.G.C terhadap C.G.S (m)
r^2 = rasio perbandingan inersia terhadap luasan
y = jarak C.G.C terhadap serat terluar
fr = modulus keruntukan (KPa)

Balok memenuhi syarat retak jika momen yang bekerja padanya tidak melampaui momen retak tahanan balok. Perhitungan kontrol momen retak tahanan dapat dihitung sebagai berikut. Dengan segmen tinjau segmen 0 saat stage 1.

A = 16,7 m^2
Ix = 44,44 m^4
r^2 = 2,66 m^2
e = 1,287 m

F = 207759,13 kN
My = -571612 kN.m
ya = 1,537 m
fr = 2091 kN/m^2

$M_{cr} = 207759{,}13\left(1{,}287 + \frac{2{,}66}{1{,}537}\right) + \frac{2091.44{,}44}{1{,}537}$

$= 683410{,}2$ kN > 571612,3 kN **(memenuhi)**

Untuk perhitungan lainnya pada stage 1 akan ditabelkan pada tabel 4.25.

Tabel 4.25 Kontrol Momen Retak Stage 1

Box	Fse (kN)	Mcr (kN.m)	My (kN.m)	CEK
S0	207759,13	683410,2	571612,3	OK
S1	207440,85	682455,9	538998,7	OK
S2	195444,17	629787,8	466814,6	OK
S3	182166,76	575320,9	401074,7	OK
S4	168490,54	521788,4	341747,2	OK
S5	154652,17	469916,6	288800,9	OK
S6	140738,96	419969,1	242204,9	OK
S7	126788,05	372055,8	201929,1	OK
S8	112935,12	326518,4	175430,1	OK
S9	98887,42	282633,9	139446,7	OK
S10	84919,60	241080,9	116533,1	OK
S11	70810,62	201345,9	85457,38	OK
S12	56742,78	163854,1	65784,28	OK
S13	42588,07	128328,2	41133,69	OK
S14	28435,01	94965,01	25716,08	OK
S15	14232,00	63662,99	9626,62	OK

4.4.1.3 Kontrol Momen Tahanan Batas

Momen tahanan batas pada box yang akan dianalisa menggunakan prinsip kesetibangan statis aksial (kopel), dimana besarnya gaya tekan batas beton (C) bernilai sama dengan gaya tarik batas pada (T), dengan menghitung lengan momen antara gaya C dan gaya T maka akan didapatkan nilai momen batas (Mu), SNI T-12-2004 pasal 4.5.1 faktor reduksi terhadap lentur dapat diambil 0,8. Berikut persamaan yang digunakan untuk perhitungan momen kopel :

$\mathrm{Mu} = \emptyset\left(\mathrm{T}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)\right)$ dan $\mathrm{a} = \frac{\mathrm{T}}{0{,}85.f'c.\mathrm{bf}}$

dimana :

Mu = momen batas (kN.m)
T = gaya prategang efektif (kN)
d = tinggi efektif penampang (m)
a = tinggi efektif daerah tekan beton (m)
bw = lebar flens bawah (m)

Perhitungan dilakukan saat stage 1 segmen 0 sebagai berikut :

T = 207759,13 kN
f'c = 70000 kN/m^2
d = 4,25 m
bf = 5,8 m
My = 536172 kN.m
a = $\frac{207759{,}13}{0{,}85.70000.5{,}8}$
= 0,602 m (gaya tekan beton hingga web badan)

T – 0,85.f’c.bf.0,5 = 207759,13 – 0,85x70000x5,8x0,5
= 84509,13 kN

Tinggi tekan pada web badan = $\frac{84509{,}13}{0{,}85.f'c.\mathrm{bw}}$
= $\frac{84509{,}13}{0{,}85.70000.0{,}5963.2}$
= 1,191 m

Mu = 0,8 x (123250x(4,25-0,25) + 84509,13 (4,25-0,5-1,191/2)
= 624570,9 kN.m > 536172 kN **(memenuhi)**

Untuk perhitungan lainnya pada stage 1 dilihat pada tabel 4.26.

Tabel 4.26 Kontrol Momen Kopel Stage 1

Box	Fse (kN)	a (m)	t web (m)	Mu (kN)	My (kN)	CEK
S0	207759,13	0,6020	1,191	624570,9	571612,3	OK

S1	207440,85	0,6011	1,186	623855,1	538998,7	OK
S2	195444,17	0,5660	0,990	596546,1	466814,6	OK
S3	182166,76	0,5279	0,798	536359,7	401074,7	OK
S4	168490,54	0,4882	0,000	501450,4	341747,2	OK
S5	154652,17	0,4481	0,000	450963,1	288800,9	OK
S6	140738,96	0,4078	0,000	401939,1	242204,9	OK
S7	126788,05	0,3674	0,000	354486,6	201929,1	OK
S8	112935,12	0,3273	0,000	308964	175430,1	OK
S9	98887,42	0,2865	0,000	264608,6	139446,7	OK
S10	84919,60	0,2461	0,000	222137,5	116533,1	OK
S11	70810,62	0,2052	0,000	180993,3	85457,38	OK
S12	56742,78	0,1644	0,000	141637,7	65784,28	OK
S13	42588,07	0,1234	0,000	103759,5	41133,69	OK
S14	28435,01	0,0824	0,000	67577,65	25716,08	OK
S15	14232,00	0,0412	0,000	32973,23	9626,62	OK

4.4.1.4 Kontrol Lendutan

Lendutan pada saat pelaksanaan adalah lendutan sementara yang diakibatkan kombinasi beban ijin baik beban prategang atau beban pelaksanaan pada jembatan yang nilainya tidak boleh melampaui lendutan yang diijinkan. Berdasarkan RSNI T-12-2004 lendutan tidak boleh melampaui L/400 untuk kantilever. Berikut merupakan perhitungan lendutan jembatan yang kemudian dibandingkan dengan lendutan yang diijinkan.

a) Akibat beban sendiri dan traveler
   Saat stage 1 dengan bantuan MIDAS CIVIL didapatkan besarnya lendutan pada ujung kantilever sebesar -0,383 m

b) Akibat tendon prategang

$$\Delta 2 = \frac{5.F_{tumpuan}.e.l^2}{48.Ec.Ix} - \frac{F_{ujung}.e.l^2}{8.Ec.Ix}$$

$$= \frac{5x207759,13x1,287x124^2}{48x39323021x44,44} - \frac{14232x1,287x124^2}{8x39323021x44,44}$$

$$= +0,225 \text{ m}$$

Lendutan total = -0,383+0,225

= -0,158 m

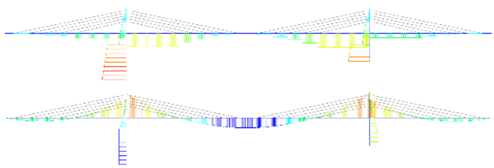
Lendutan ijin = L/400

= 65/400

= 0,1625 m > 0,158 m **(Memenuhi)**

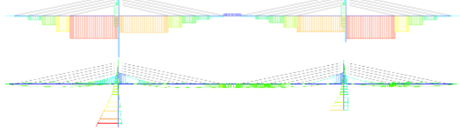
4.4.2 Perencanaan Tendon Menerus

Perencanaan tendon menerus untuk pembebanan penuh mempertimbangkan tendon pada pembebanan pelaksanaan yang sudah diberikan, kekurangan gaya prategang akan diberikan saat menahan beban penuh, khususnya pada daerah lapangan untuk menentukan tendon daerah tumpuan maupun lapangan akan digunakan program bantu MIDAS CIVIL dengan mempertimbangkan kondisi pembebanan yang paling kritis. Berikut adalah gaya dalam saat pemasangan closure untuk lebih jelasnya dapat melihat gambar 4.20.



Gambar 4.20 Gaya Aksial Dan Momen Saat Closure

Selain mempertimbangkan kondisi saat pemasangan closure, harus ditinjau juga saat jembatan dibebani oleh beban hidup. Yaitu dengan memperhatikan momen maximum dan minumun dari kombinasi beban Kuat, Ekstream, dan Layan. Serta mempertimbangkan kondisi setelah *stay* kabel jembatan diberi gaya *pretension*. Berikut adalah gaya dalam envelope dari seluruh kombinasi yang dapat dilihat gambar 4.21.



Gambar 4.21 Gaya Aksial Dan Momen Envelope

Berikut adalah gaya dalam lantai kendaraan saat pemasangan closure dan envelope setelah *stay* kabel jembatan telah diberikan *pretension* yang dapat dilihat pada tabel 4.27.

Tabel 4.27 Gaya dalam lantai kendaraan

Box		Closure		Envelope	
		Axial (kN)	Momen-y (kN)	Axial (kN)	Momen-y (kN)
SIDE	S16	38.69	17504.95	499.65	26418.81
	S15	-162.47	35815.35	1377.04	54135.98
	S14	-173.64	47905.88	2382.26	70554.31
	S13	-959.45	55054.92	-12271.6	92647.92
	S12	-971.06	55473.98	-12288.8	94213.77
	S11	-2496.8	52145.9	-32043.9	114173.1
	S10	-2508.89	41414.9	-32064.8	105892.4
	S9	-4780.65	28273.01	-56384.8	124455
	S8	-4793.26	6996.4	-56416.6	109182.2
	S7	-7612.29	-15558.6	-81502.3	123526.9
	S6	-7625.47	-46850.7	-81524.5	104363.3
	S5	-7639.16	-84470.7	-81547.2	77428.8
	S4	-7653.22	-128449	-81570.6	-67589.1
	S3	-7667.67	-178817	-81594.6	-105774
	S2	-7682.53	-235606	-81619.2	-167269
	S1	-7697.8	-298848	-81644.5	-239802
	S0	-21740.9	-417864	-106929	-386306
MID	S1	-21910.7	-377043	-107058	-336450
	S2	-21895.1	-301003	-107039	-248543
	S3	-21879.9	-231410	-107021	-171667
	S4	-21865.2	-168232	-107003	-105781
	S5	-21850.8	-111439	-106985	69932.93

	S6	-21836.8	-60999.2	-106968	104930.6
	S7	-21823.4	-16882.2	-106952	129977
	S8	-15397.9	15453.26	-90237.2	124293
	S9	-15385	53014.5	-90213.8	144250.3
	S10	-9857.96	79833.29	-72932.2	133128.4
	S11	-9845.6	109839.5	-72916.7	144029.1
	S12	-5782.07	130534.4	-56390.6	127293.4
	S13	-5770.19	151656	-56375.5	128067.7
	S14	-2861.44	164545.6	-41358.6	104765.3
	S15	-2850	175820.9	-41344	110350.8
	S16	-528.57	179446.9	-27590.1	100325.3

Direncanakan menggunakan tendon / kabel jenis strand *seven wires stress relieved* (7 kawat untaian). Dengan mengacu pada tabel VSL tipe ASTM A 416-06 Grade 270, berikut adalah jenis dan karakteristik tendon yang digunakan :

- Diameter : 15,24 mm
- Luas nominal (Ap) : 140 mm^2
- Kuat nominal tarik : 1860 MPa
- Min. breaking load : 260,7 kN
- Modulus elastisitas : 200 Gpa

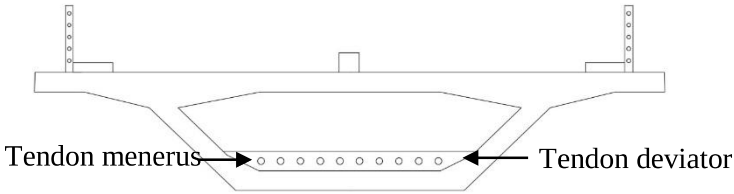
Unit tendon yang digunakan sebagai tendon menerus berupa 6-55-55 dengan spesifikasi sebagai berikut :

- Steal area : 7700 mm^2
- Min. breaking load : 14339 kN

F_{ijin} = 0,9 x min. breaking load
= 0,9 x 14339
= 12905,1 kN

fs = F_{ijin}/Ap
= 12905100/7700
=1675,99 MPa

Direncanakan letak tendon menerus jembatan diletakan secara eksternal pada bagian dalam box girder dengan bantuan tendon deviator. Untuk lebih jelasnya dapat melihat gambar 4.22.



Gambar 4.22 Letak tendon konstruksi

Dengan mengambil nilai e pada joint sebagai berikut :

$e = yb – 0{,}75$

$\sigma_{\text{ijin tekan}} = 0{,}45 \times f'c$

$= 0{,}45 \times 70$ MPa

$= 31{,}5$ Mpa

$\sigma_{\text{ijin tarik}} = 0{,}25 \times \sqrt{f'c}$

$= 4{,}183$ Mpa

Berikut adalah jumlah kebutuhan tendon konstruksi dan tendon menerus pada tiap-tiap segmen yang tertera pada tabel 4.28.

Tabel 4.28 Kebutuhan Tendon

<table>
<tr><td colspan="2" rowspan="2">Box</td><td>Tendon atas</td><td>Tendon bawah</td></tr>
<tr><td>6-31-31</td><td>6-55-55</td></tr>
<tr><td rowspan="7">SIDE</td><td>S16</td><td></td><td></td></tr>
<tr><td>S15</td><td>2</td><td>4</td></tr>
<tr><td>S14</td><td>4</td><td>4</td></tr>
<tr><td>S13</td><td>6</td><td>5</td></tr>
<tr><td>S12</td><td>8</td><td>5</td></tr>
<tr><td>S11</td><td>10</td><td>5</td></tr>
<tr><td>S10</td><td>12</td><td>5</td></tr>
</table>

	S9	14	5
	S8	16	5
	S7	18	5
	S6	20	4
	S5	22	4
	S4	24	
	S3	26	
	S2	28	
	S1	30	
	S0	30	
MID	S1	30	
	S2	28	
	S3	26	
	S4	24	
	S5	22	
	S6	20	4
	S7	18	6
	S8	16	6
	S9	14	6
	S10	12	6
	S11	10	6
	S12	8	6
	S13	6	8
	S14	4	8
	S15	2	10
	S16		10

Sebelum memperhitung kemampuan penampang box akibat tendon menerus, perlu dilakukan kontrol kehilangan prategang akibat beban aksial dan momen pada tendon konstruksi saat pemasangan closure setelah beban hidup masuk, berikut contoh saat pemasangan closure pada box 15 pada *midspan* :

1. Akibat perpendekan elastis beton Δes

   $\Delta es = K_{ES} \cdot \frac{E_s}{E_c} \cdot f_{cir}$

   $= 0{,}5 \text{ x } \frac{195000}{39323{,}021} \text{ x } -6{,}205$

   $= -15{,}384$ Mpa

2. Akibat gesekan Δfs1

   $\Delta fs1 = (-K.L - \mu.\alpha).fs$

   $= (0{,}0045 \text{ x } 4 – 0{,}2 \text{ x } 0).1675{,}99$

   $= 30{,}168$ Mpa

3. Akibat slip angkur Δfs2

   $\Delta\alpha = \frac{fs\,.L}{Es}$

   $= \frac{1675{,}99\,.116000}{195000}$

   $= 997$ mm

   $\Delta fs2 = \frac{2{,}5}{\Delta\alpha} \cdot fs$

   $= \frac{2{,}5}{997} \text{ x } 1675{,}99$

   $= 3{,}93$ MPa

a) Tegangan efektif pada tendon

   $fse = fs - \Delta es - \Delta fs1 - \Delta fs2$

   $= 1675{,}99 + 15{,}384 – 30{,}168 – 3{,}93$

   $= 1657{,}276$ Mpa

b) Gaya prategang efektif pada tendon

   Fse = jumlah tendon x fse x Ap

   = 2x 1657,276 x 4340

   = 14385,16 kN

Setelah didapatkan tegangan efektif tendon konstruksi dilakukan kontrol penampang akibat tendon menerus, sebagai contoh berikut adalah perhitungan pada box 15 saat pemasangan closure :

$$\sigma_{atas} = \frac{F}{A} + \frac{Fatas.e.ya}{Ix} - \frac{Fbawah.e.ya}{Ix} + \frac{My.ya}{Ix}$$

$$= \frac{2850+14385,16+10*12905,1}{15,875} + \frac{14385,16x0,844x1,094}{18,915} - \frac{10*12905,1x1,322x1,094}{18,915} + \frac{175820,9x1,094}{18,915}$$

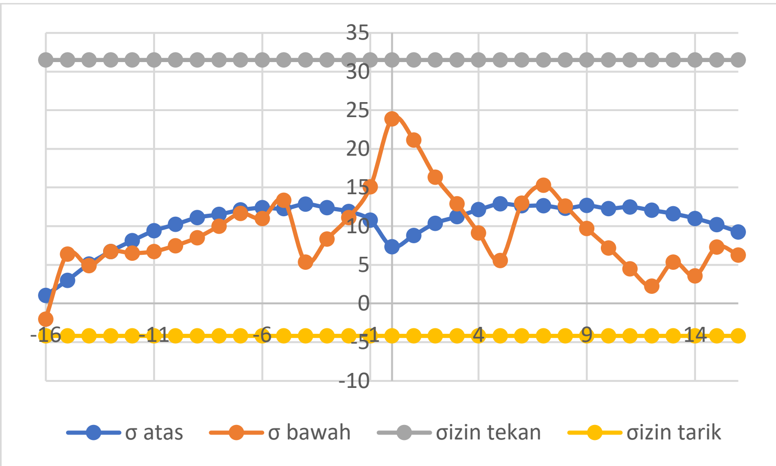
= 10,22MPa < 31,5 MPa **(memenuhi)**

$$\sigma_{bawah} = \frac{F}{A} - \frac{Fatas.e.yb}{Ix} + \frac{Fbawah.e.ya}{Ix} - \frac{My.yb}{Ix}$$

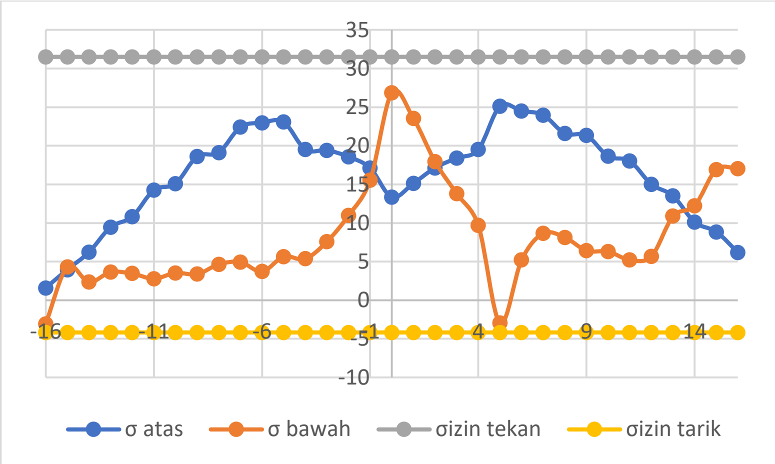
$$= \frac{2850+14385,16+10*12905,1}{15,875} - \frac{14385,16x0,844x2,072}{18,915} + \frac{10*12905,1x1,322x2,072}{18,915} - \frac{175820,9x2,072}{18,915}$$

= 7,32 MPa > -4,183 MPa **(memenuhi)**

Berikut adalah gambar tegangan pada tiap-tiap segmen yang tertera pada gambar 4.23 dan 4.24.



Gambar 4.23 Tegangan Box Girder Saat Closure



Gambar 4.24 Tegangan Box Girder Saat Envelope

4.4.2.1 Kehilangan Prategang

Kontrol kehilangan kali ini diberikan contoh perhitungan saat pemasangan closure dan ditinjau box 15 pada *midspan*.

1. Akibat perpendekan elastis beton Δes

$$\Delta es = K_{ES} \cdot \frac{E_s}{E_c} \cdot f_{cir}$$

$$= 0{,}5 \text{ x } \frac{195000}{39323{,}021} \text{ x } 4{,}668$$

$$= 11{,}575 \text{ Mpa}$$

2. Akibat gesekan Δfs1

$$\Delta fs1 = (-K.L - \mu.\alpha).fs$$

$$= (0{,}0045 \text{ x } 4 – 0{,}2 \text{ x } 0).1675{,}99$$

$$= 30{,}168 \text{ Mpa}$$

3. Akibat slip angkur Δfs2

$$\Delta\alpha = \frac{fs \, .L}{Es}$$

$$= \frac{1675{,}99 \, .14000}{195000}$$

$$= 120{,}327 \text{ mm}$$

Δfs2 $= \frac{2,5}{\Delta\alpha} \cdot fs$

$= \frac{2,5}{120,327}$ x 1675,99

= 34,82 Mpa

4. Akibat rangkak CR

   Salah satu sifat beton adalah dapat mengalami tambahan regangan akibat beban tetap (mati) seiring dengan semakin bertambahnya waktu. Metode umum untuk memperhitungkan rangkak pada beton adalah dengan memasukkan kedalam perhitungan hal-hal berikut ini : Perbandingan volume terhadap permukaan, umur beton pada saat prategang, kelembaban relative dan jenis beton (beton ringan atau normal). Kehilangan gaya prategang akibat rangkak untuk komponen struktur dengan tendon terekat dapat dihitung dengan persamaan sebagai berikut (untuk beton dengan berat normal) :

   CR $= K_{cr} \frac{E_s}{E_c}$(fcir-fcds)

   $= 1,6 \frac{200000}{39323,021}$(4,668+12,11)

   = 133,13 Mpa

   dimana :

   K_{cr} = 1,6 unutk komponen struktur pasca tarik
   fcds = tegangan beton pada C.G.S akibat beban
   fcir = tegangan beton pada C.G.S akibat tendon
   Es = modulus elastisitas tendon
   Ec = modulus elastisitas beton

5. Akibat susut SH

   Susut pada beton dipengaruhi oleh berbagai faktor seperti rangkak, perbandingan antara volume dan permukaan, kelembaban relatif, dan waktu dari akhir perawatan sampai dengan bekerjanya gaya prategang. Persamaan yang dipakai dalam memperhitungkan kehilangan pratekan akibat susut pada beton adalah :

SH = $8,2.10^{-6}$ x K_{sh} x Es x (1-0,06xV/S) x (100-RH)
= $8,2.10^{-6}$ x 0,77 x 195000 x (1-0,06x221,58/487,317) x (100-80)
= 23,953 MPa

dimana :

K_{sh} = 0,77 (tabel T.Y.Lin hal 88)
V = volume (m^3)
S = selimut (m^2)
RH = kelembaban udara diambil 80%

6. Akibat relaksasi RE

Sebenarnya balok pratekan mengalami perubahan regangan baja yang konstan di dalam tendon bila terjadi rangkak yang tergantung pada waktu. Akibat perpendekan elastis (ES), serta kehilangan gaya pratekan yang tergantung pada waktu yaitu CR dan SH, maka akan mengakibatkan terjadi pengurangan yang kontinu pada tegangan tendon. Oleh karena itu untuk memperkirakan kehilangan gaya pratekan akibat pengaruh tersebut digunakan perumusan sebagai berikut :

RE = (K_{re} – J.(SH + CR + ES)).C
= (128 – 0,14.(133,13 + 29,95 -1,156)) 0,28
= 29,23 MPa

dimana :

K_{re} = 128 MPa (tabel 4-5 T.Y.Lin)
J = 0,14 (tabel 4-5 T.Y.Lin)
C = 0,28 (tabel 3.4 T.Y.Lin)

a) Tegangan efektif pada tendon

fse = *fs* - Δes - Δfs1 - Δfs2 - CR – SH - RE
= 1675,99 + 11,6 – 30,17 – 34,82 – 29,95 – 133,13 – 29,23
= 1413,11 Mpa

b) Gaya prategang efektif pada tendon

Fse = jumlah tendon x fse x Ap
= 10 x 1413,11 x 7700

= 108809,4 kN

c) Kontrol gaya prategang efektif

$$\sigma_{atas} = \frac{F}{A} + \frac{Fatas.e.ya}{Ix} - \frac{Fbawah.e.ya}{Ix} + \frac{My.ya}{Ix}$$

$$= \frac{2850+14385,16+108809,4}{15,875} + \frac{14385,16x0,844x1,094}{18,915} - \frac{108809,4\ x1,322x1,094}{18,915} + \frac{175820,9x1,094}{18,915}$$

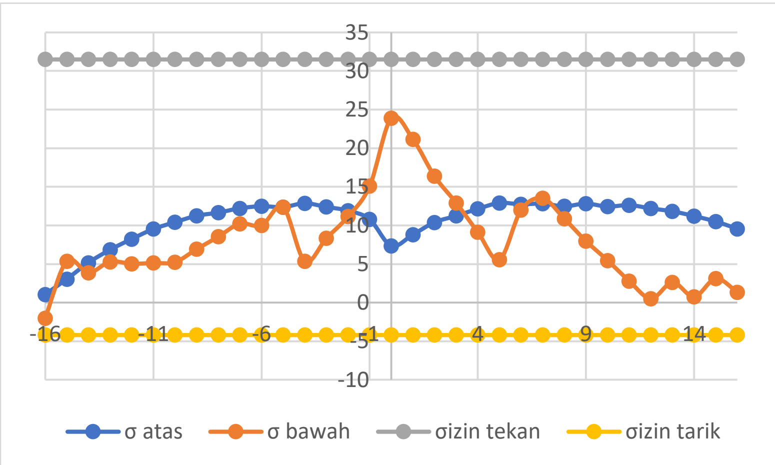
= 10,49MPa < 31,5 MPa **(memenuhi)**

$$\sigma_{bawah} = \frac{F}{A} - \frac{Fatas.e.yb}{Ix} + \frac{Fbawah.e.ya}{Ix} - \frac{My.yb}{Ix}$$

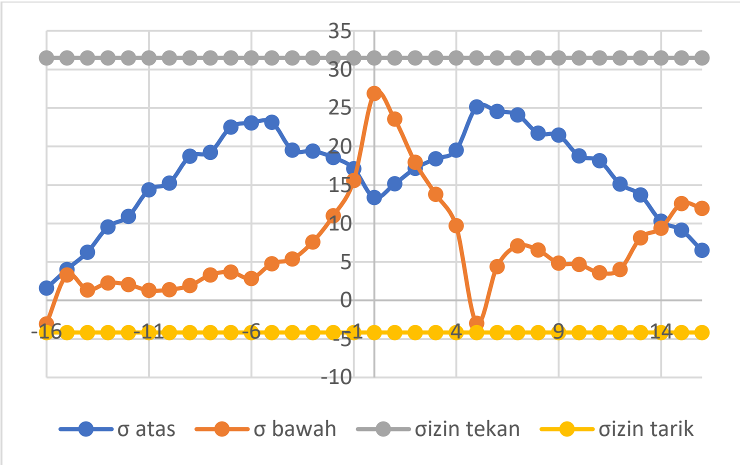
$$= \frac{2850+14385,16+108809,4}{15,875} - \frac{14385,16x0,844x2,072}{18,915} + \frac{108809,4\ x1,322x2,072}{18,915} - \frac{175820,9x2,072}{18,915}$$

= 3,11 MPa > -4,183 MPa **(memenuhi)**

Berikut adalah gambar tegangan efektif pada tiap-tiap segmen yang tertera pada gambar 4.25 dan 4.26.



Gambar 4.25 Tegangan efektif box girder saat closure



Gambar 4.26 Tegangan efektif box girder saat envelope

4.4.2.2 Kontrol Momen Retak

Balok memenuhi syarat retak jika momen yang bekerja padanya tidak melampaui momen retak tahanan balok. Perhitungan kontrol momen retak tahanan dapat dihitung sebagai berikut, dengan segmen box 15 saat pemasangan closure.

A = 15,87 m^2 F = 108809,4 kN
Ix = 18,92 m^4 My = 175820,9 kN.m
r^2 = 1,19 m^2 yb = 2,072 m
e = 1,322 m fr = 4183 kN/m^2

$$M_{cr} = F\left(e + \frac{r^2}{y}\right) + \frac{fr.Ix}{y}$$

$$= 108809{,}4\left(1{,}322 + \frac{1{,}19}{2{,}072}\right) + \frac{4183.18{,}92}{2{,}072}$$

= 244621,2 kN > 175820,9 kN **(memenuhi)**

4.4.2.3 Kontrol Momen Tahanan Batas

Momen tahanan batas pada box yang akan dianalisa menggunakan prinsip kesetibangan statis aksial (kopel), dimana besarnya gaya tekan batas beton (C) bernilai sama dengan gaya tarik batas pada (T), dengan menghitung lengan momen antara gaya C dan gaya T maka akan didapatkan nilai momen batas (Mu), SNI T-12-2004 pasal 4.5.1 faktor reduksi terhadap lentur dapat diambil 0,8. Pada segmen box 15 saat pemasangan closure sebagai berikut :

T = 108809,4 kN

f'c = 70000 kN/m^2

d = 2,417 m

bf = 16 m

My = 175820,9 kN.m

a $= \frac{T}{0{,}85.f'c.\text{bf}}$

$= \frac{108809{,}4}{0{,}85.70000.16}$

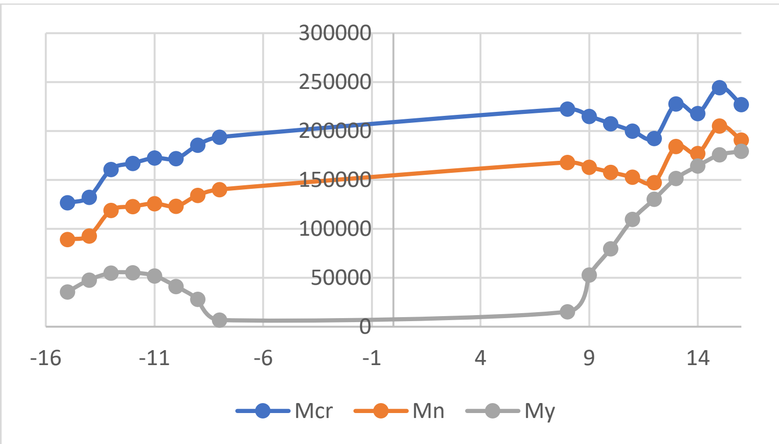
= 0,1143 m

Mu $= \emptyset\left(T\left(d - \frac{a}{2}\right)\right)$

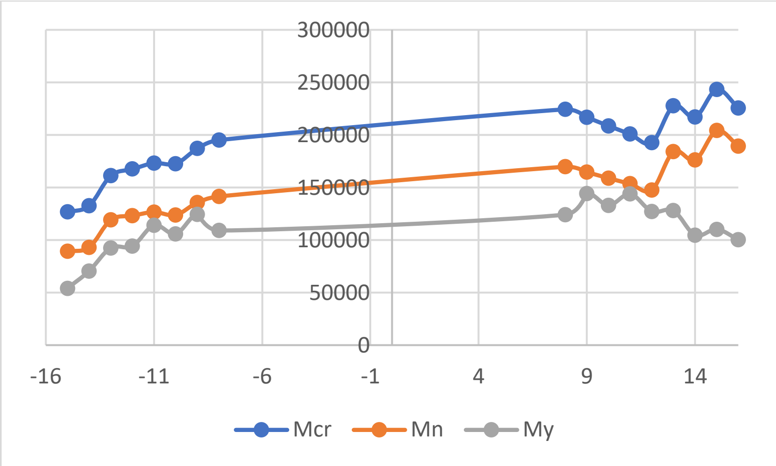
= 0,8 x (108809,4x(2,417-0,1143/2)

= 205390,3 kN.m > 175820,9 kN **(memenuhi)**

Berikut adalah gambar momen pada tiap-tiap segmen yang tertera pada gambar 4.27 dan 4.28.



Gambar 4.27 Kontrol Momen Saat Closure



Gambar 4.28 Kontrol Momen Saat Envelope

4.4.2.4 Kontrol Lendutan

Lendutan pada saat beban penuh adalah lendutan jangka panjang akibat kombinasi beban ijin baik beban prategang, beban mati maupun beban hidup. Lendutan tersebut tidak boleh melampaui lendutan yang di ijinkan. Berdasarkan RSNI T-122004 lendutan tidak boleh melampaui L/800 . Berikut merupakan perhitungan lendutan jembatan kemudian dibandingkan dengan lendutan yang diijinkan. Persamaan lendutan untuk kantilever adalah sebagai berikut (McCormac, 2003).

a) Akibat beban sendiri dan traveler
   Dengan bantuan MIDAS CIVIL didapatkan besarnya lendutan pada tengah bentang sebesar -0,205 m

b) Akibat tendon menerus

$$\Delta 2 = \frac{5.F_{tumpuan}.e.l^2}{48.Ec.Ix} - \frac{F_{ujung}.e.l^2}{8.Ec.Ix}$$

$$= \frac{5x104508,3x1,258x12^2}{48x39323021x17,518} - \frac{1548,59x1,258x12^2}{8x39323021x17,518}$$

$$= +0,003 \text{ m}$$

Lendutan total = -0,205+0,003

= -0,202 m

Lendutan ijin = L/800

= 260/800

= 0,325 m > 0,202 m **(Memenuhi)**

4.4.3 Kontrol Torsi

Karena kekuatan geser beton yang tinggi digabungkan dengan kekuatan tarik yang rendah, kehancuran balok beton akibat puntir jarang disebabkan oleh tegangan geser, melainkan lebih disebabkan oleh tegangan tarik utama yang diakibatkan oleh tegangan geser. Pada waktu tegangan tarik utama mencapai kekuatan tarik batas beton, retak mulai terjadi dan penampang dapat runtuh seketika tanpa banyak peringatan. Penambahan senngkang tertutup dan tulangan longitudinal dapat menambah kekuatan dan daktilitas, tetapi bentuk retak akibat puntir secara drastis mempengaruhi respons balok terhadap setiap penambahan momen puntir.

Bertentangan dengan ragam kehancuran akibat puntir, balok beton prategang di bawah pengaruh lentur umumnya runtuh secara perlahan-lahan dan memiliki kekuatan cadangan serta daktilitas setelah retak-retak pertama terlihat. Hal ini menjadi jelas bila disadari bahwa kehancuran akibat lentur tergantung pada tegangan tarik dan regangan baja, bersamaan dengan tegangan tekan dan regangan beton. Sedangkan kekuatan puntir sebuah balok tanpa tulangan badan untuk puntir akan lenyap bila batas tarik beton dicapai dan tidak ada daktilitas beton akibat tegangan Tarik Perhitungan torsi ini pada segmen ke 16 (perhitungan ini akibat Kombinasi Extrem 1)

$$\eta_i = \frac{0{,}35}{\left[0{,}75+\left(\frac{x1}{y1}\right)\right]}$$

- Pelat atas

x1 = tebal pelat atas = 500 mm

y1 = lebar pelat atas = 16000 mm
ɳ1 = 0,448

- Pelat web

x2 = 721 mm
y2 = 2000 mm
ɳ2 = 0,315

- Pelat bawah

x3 = 500 mm
y3 = 5800 mm
ɳ3 = 0,418

Konstanta torsi $= \sum \eta_i . x_i{}^2 y_i$
$= 0{,}448.500^2.16000 + 2.0{,}315.721^2.2000 + 0{,}418.500^2.5800$
$= 3.053.099.660$

$$T_{cr} = 6\sqrt{f'c}.\sqrt{1+\frac{10.\left(\frac{F}{A}\right)}{f'c}}.\left(\sum \eta_i . x_i{}^2 y_i\right)$$

$$= 6\sqrt{70}.\sqrt{1+\frac{10.\left(\frac{129383{,}4}{15{,}71.1000}\right)}{80}}.(3053099660)$$

$= 2{,}183 \times 10^{11}$ Nmm

Reduksi T_{cr}

$0{,}7 \times T_{cr} = 1{,}528 \times 10^{11}$

Torsi ijin Berdasarkan SNI T-12-2004 Pasal 5.4.5 Tulangan puntir tidak diperlukan apabila :

$\frac{T_u}{\emptyset T_{cr}} < 0{,}25$

$\frac{1419980000 \text{ Nmm}}{1{,}528x10^{11}\text{Nmm}} < 0{,}25$

$0{,}00929 < 0{,}25$

Dari perhitungan diatas dapat disimpulkan bawha box girder tidak memerlukan tulangan torsi.

4.4.4 Perencanaan Penulangan Box Girder

Untuk penulangan box girder momen di ambil dari output SAP2000 dengan pembebanan dan permodelan sebagai berikut :

Case 1 :

- Beban Terbagi Rata (BTR)

BTR = 9 kPa x 1 m

= 9 kN/m

- Beban Garis Terpusat (BTR)

BGT = BGT x (1+FBD)

= 49 kN/m x (1+40%)

= 49 x 1,4

= 68,6 kN/m

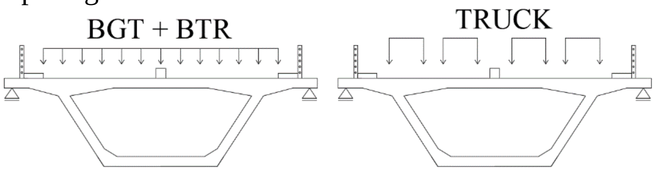
Case 2

Beban Truck = 112,5 kN x (1+FBD)

= 112,5 x 1,3

= 146,25 kN / 1 roda

Yang akan dibebankan pada lantai kendaraan secara melintang. Kemudian beban-beban tersebut di inputkan kepada SAP2000 untuk memperoleh hasil gaya dalam yang permodelannya dapat dilihat pada gambar 4.29.



Gambar 4.29 Permodelan box girder

Berdasarkan hasil analisa SAP2000 didapatkan gaya dalam sebagai berikut yang ditabelkan pada tabel 4.29.

Tabel 4.29 Gaya dalam box girder

	Mu+ (kN.m)	Mu- (kN.m)	Vu (kN)
Flens atas	2061,85	-700,65	1115,156
Web	1958,56	-429,3	700,87
Flens bawah	0	-429,3	44,36

1. Penulangan flens arah melintang

   Data perencanaan :

f'c	= 70 MPa
fy	= 400 MPa
cover	= 50 mm
tulangan	= D36
d	= 500-50-(0,5x32)
	= 434 mm
b	= 1000 mm
ϕ	= 0,9

   Tulangan minimum RSNI T-12-2004

   $$\rho_{min} = \frac{\sqrt{f'c}}{4.fy} = \frac{\sqrt{70}}{4.400} = 0{,}00523$$

   tidak kurang dari

   $$\rho_{min} = \frac{1{,}4}{fy} = \frac{1{,}4}{400} = 0{,}0035$$

   Berdasarkan RSNI T-12-2004 pasal 5.1.1.5c sebagai alternatif, untuk komponen struktur yang besar dan masif, luas tulangan yang diperlukan pada setiap penampang positif atau negaatif. Paling sedikit harus sepertiga lebih besar dari yang diperlukan berdasarkan analisis.

   Tulangan maksimum RSNI T-12-2004 pasal 5.1.1.6

   $$\beta 1 = 0{,}85 - 0{,}05\frac{f'c-28}{7} \quad > 0{,}65$$

   $$= 0{,}85 - 0{,}05\frac{70-28}{7} \quad > 0{,}65$$

   $$= 0{,}55 \text{ digunakan } 0{,}65$$

   $$\rho b = \frac{0{,}85.\beta 1.f'c}{fy}\left(\frac{600}{600+fy}\right)$$

   $$= \frac{0{,}85.0{,}65.70}{400}\left(\frac{600}{600+400}\right)$$

   $$= 0{,}058$$

   $$\rho_{max} = 0{,}75x\ \rho b$$

   $$= 0{,}0435$$

   $$Mn = Mu/\ \phi$$

$= 2061,85/0,9$

$= 2290,94$ kN.m

Rn $= \frac{Mn}{bxd^2}$

$= \frac{2290940000}{1000x434^2}$

$= 12,16$ MPa

m $= \frac{fy}{0,85\ f'c}$

$= \frac{400}{0,85\ 70}$

$= 6,72$

$\rho_{perlu} = \frac{1}{m}\left(1 - \sqrt{\frac{2.m.Rn}{fy}}\right)$

$= \frac{1}{6,72}\left(1 - \sqrt{1 - \frac{2.6,72.12,16}{400}}\right)$

$= 0,0343$ **(Oke)**

As perlu $= 0,0343 \times 1000 \times 434$

$= 14886,2\ mm^2$

s perlu $= \frac{\frac{1}{4}.\pi.D^2.b}{As}$

$= \frac{\frac{1}{4}.\pi.32^2.1000}{148866,2}$

$= 54$

Pakai 2D32 – 100

Kontrol momen

T $= 20 \times As \times fy$

$= 20 \times 803,84 \times 400$

$= 6430720$ N

a $= \frac{T}{0,85.f'c.bf}$

$= \frac{6430720}{0,85.70.1000}$

$= 108,08$ mm

$Mu = \emptyset\left(T\left(d - \frac{a}{2}\right)\right)$

$= 0{,}9\left(6430720\left(434 - \frac{108{,}08}{2}\right)\right)$

$= 2199{,}07$ kN > Mu **(Memenuhi)**

Maka digunakan 2D32 – 100

Untuk perhitungan tulangan lainnya akan ditabelkan pada tabel 4.30.

Tabel 4.30 Kebutuhan tulangan box girder

	Tul +	Tul -
Flens atas	2D32 - 100	D32 - 150
Web	2D32 - 100	D32 - 150
Flens bawah	D32 - 300	D32 - 150

2. Penulangan flens arah memanjang

Dikarenakan momen arah longitudinal dipikul seluruhnya oleh tendon dan untuk mengantisipasi susut dan suhu, maka digunakan tulangan minimum susut.

ρ_{min} = 0,002

As perlu = 0,002 x 1000 x 434

= 868 mm^2

Digunakan D22

s perlu $= \frac{\frac{1}{4}.\pi.D^2.b}{As}$

$= \frac{\frac{1}{4}.\pi.22^2.1000}{868}$

= 437,72

Maka digunakan D22 - 300

4.5 Analisa Kabel

4.5.1 Gaya Prestension Kabel

Dalam pelaksanaannya masing-masing kabel diberi gaya tarik (*pretension*) terlebih dahulu. Hal ini bertujuan untuk mengatur posisi gelagar agar berada pada posisi *center line* sebelum menerima beban hidup. Apabila gaya tarik ini tidak diberikan pada kabel, gelagar akan melendut terlebih dahulu sebelum menerima beban hidup. Hal ini terjadi karena adanya beban mati struktur yang mengakibatkan deformasi kabel dan menyebabkan turunnya gelagar utama.

Dengan bantuan program bantu MIDAS CIVIL dapat dianalisa besarnya gaya Tarik masing-masing kabel dengan fitur *unknown load factor*. Berikut langkah-langkah Analisa gaya Tarik kabel :

1. Memberi gaya tarik pada setiap kabel sebesar 1 unit.
2. Memberi batasan deformasi pada gelagar. Hal ini bertujuan untuk memberikan input pada program agar gelagar tidak melendut sesuai dengan Batasan yang diberikan saat *stressing*. Besaran batasan yang diberikan sebesar ± 1 cm.
3. Menentukan beban apa saja yang mempengaruhi kondisi saat *stressing*. Dalam hal ini beban mati struktur dan beban mati tambah.
4. Melakukan iterasi dengan program bantu MIDAS CIVIL.

Berikut hasil dari iterasi gaya tarik awal kabel dapat dilihat pada tabel 4.31.

Tabel 4.31 Iterasi gaya tarik kabel

Kabel	Prestress (kN)	Kabel	Prestress (kN)
1	10831,04	6	7318,119
2	11019,66	7	8028,552
3	9206,389	8	8972,431
4	5413,625	9	9658,078
5	273,358	10	9755,877

Perencanaan kebutuhan kabel awal direncanakan dengan cara menghitung luasan tendon berdasarkan gaya *prestress* diatas dengan kemampuan 50% dari *breaking load* kabel. Untuk lebih jelasnya dapat melihat tabel 4.32.

Fu kabel = 1860 MPa

Fu ijin = 0,5 x 1860 MPa
= 930 MPa
= 0.93 kN/mm^2

Tabel 4.32 Kebutuhan jumlah kabel

Kabel	Prestress	Fijin (kN/mm^2)	Asc (mm^2)	n butuh	n pakai
1	10831,04	0,93	11646,28	84	115
2	11019,66	0,93	11849,09	85	107
3	9206,389	0,93	9899,343	71	84
4	5413,625	0,93	5821,102	42	52
5	273,358	0,93	293,9333	3	16
6	7318,119	0,93	7868,945	57	59
7	8028,552	0,93	8632,852	62	64
8	8972,431	0,93	9647,775	69	81
9	9658,078	0,93	10385,03	75	103
10	9755,877	0,93	10490,19	75	114

4.5.2 Analisa Kabel Aktual

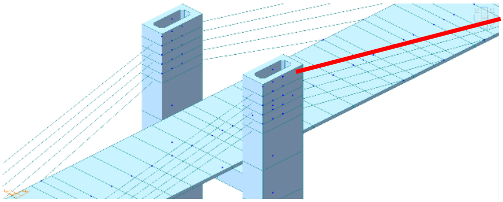
Setelah diperoleh nilai A_{aktual} dan gaya *prestress* kabel *dari* perhitungan sebelumnya. Dilakukan kontrol gaya kabel akibat beban-beban lain yang bekerja. Untuk lebiih jelasnya dapat melihat tabel 4.33.

Tabel 4.33 Kontrol kemampuan kabel aktual

Kabel	Load	Force (kN)	n	A kabel (mm^2)	Fijin (kN/mm^2)	Pn (kN)	Cek
1	Ekstrem 1	14045,48	115	16100	1,116	17967,6	OK
2	Ekstrem 1	13156,96	107	14980	1,116	16717,68	OK
3	Ekstrem 1	10487,3	84	11760	1,116	13124,16	OK
4	Ekstrem 1	6051,602	52	7280	1,116	8124,48	OK
5	Ekstrem 1	441,8367	16	2240	1,116	2499,84	OK
6	Ekstrem 1	8028,083	59	8260	1,116	9218,16	OK

7	Ekstrem 1	8972,054	64	8960	1,116	9999,36	OK
8	Ekstrem 1	10452,15	81	11340	1,116	12655,44	OK
9	Ekstrem 1	12059,28	103	14420	1,116	16092,72	OK
10	Ekstrem 1	13295,47	114	15960	1,116	17811,36	OK

Selain harus mampu menahan beban-beban yang terjadi, suatu jembatan betipe kabel harus juga mampu menahan gaya yang terjadi bila terjadi 1 kabel putus pada jembatan seperti yang tertera pada gambar 4.30. Sedangkan untuk control kemampuan kabel jembatan bila 1 kabel putus dapat melihat tabel 4.34.



Gambar 4.30 Jembatan bila putus 1 kabel

Tabel 4.34 Kontrol kemampuan kabel putus 1

Kabel	Load	Force (kN)	n	A kabel (mm^2)	Fijin (kN/mm^2)	Pn (kN)	Cek
1	Ekstrem 1	14088,01	115	16100	1,116	17967,6	OK
2	Ekstrem 1	13194,54	107	14980	1,116	16717,68	OK
3	Ekstrem 1	10517,68	84	11760	1,116	13124,16	OK
4	Ekstrem 1	6071,493	52	7280	1,116	8124,48	OK
5	Ekstrem 1	448,2934	16	2240	1,116	2499,84	OK
6	Ekstrem 1	8085,353	59	8260	1,116	9218,16	OK
7	Ekstrem 1	9043,854	64	8960	1,116	9999,36	OK
8	Ekstrem 1	10538,78	81	11340	1,116	12655,44	OK
9	Ekstrem 1	12172,71	103	14420	1,116	16092,72	OK
10	Ekstrem 1	13427,99	114	15960	1,116	17811,36	OK

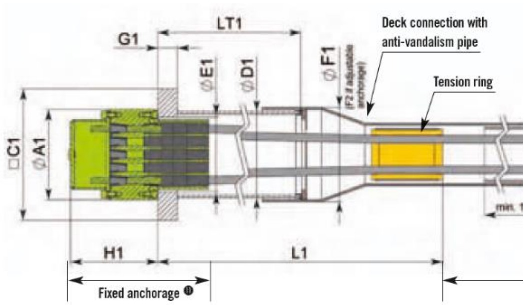
Dari kontol diatas dapat disimpulkan bahwa untuk kebutuhan jumlah strand kabel yang dipakai sudah mampu menahan gaya tarik masksimum yang terjadi. Dan juga masih mampu menahan gaya tarik maksimum setelah terjadi putusnya 1 kabel pada jembatan.

4.5.3 Analisa Blok Angkur Pada Gelagar

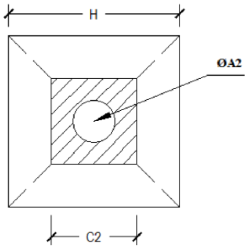
Angkur kabel dipasang sesuai jumlah strand dan gaya stressing nya yang telah dihitung. Perencanaan ini meliputi cek tegangan beton saat stressing serta kebutuhan tulangan pencar (bursting) dan tulangan pecah (spalling). Berikut adalah nilai gaya tarik dan detail serta spesifikasi angker yang digunakan dapat dilihat pada tabel 4.35 dan 4.36 serta gambar 4.31 dan 4.32.

Tabel 4.35 Kuat nominal kabel

Kabel	Pn (kN)	Kabel	Pn (kN)
1	17967,6	6	9218,16
2	16717,68	7	9999,36
3	13124,16	8	12655,44
4	8124,48	9	16092,72
5	2499,84	10	17811,36



Gambar 4.31 Detail angkur VSL SSI 2000



Gambar 4.32 Notasi dimensi angkur

Tabel 4.36 Spesifikasi teknis angkur VSL SSI 2000

			FIXED ANCHORAGE						
CABLE UNIT	NUMBER OF STRANDS	BREAKING LOAD AT 100% GUTS kN	ØA1 mm	C1 mm	ØD1/thk mm/mm	ØE1 mm	ØF1 mm	G1 mm	H1 mini mm
6-12	12	3,348	185	260	177.8/4.5	150	190	35	205
6-19	19	5,301	230	335	219.1/6.3	190	233	50	220
6-22	22	6,138	250	355	219.1/6.3	205	233	50	220
6-31	31	8,649	280	415	244.5/6.3	230	260	60	245
6-37	37	10,323	300	455	273/6,3	255	286	70	270
6-43	43	11,997	340	505	323.9/7.1	285	337	75	275
6-55	55	15,345	380	550	323.9/7.1	310	337	75	295
6-61	61	17,019	380	585	355.6/8	330	370	85	310
6-73	73	20,367	430	650	406.4/8.8	370	420	95	330
6-85	85	23,715	430	685	406.4/8.8	370	420	110	360
6-91	91	25,389	480	730	457/10	420	470	110	370
6-109	109	30,411	495	775	457/10	420	470	120	380
6-127	127	35,433	550	845	508/11	475	525	130	430
6-139	139	38,781	570	900	520/12	480	540	135	440
6-151	151	42,129	590	920	559/12.5	490	550	140	460
6-169	169	47,151	630	970	585/14	510	580	150	480
6-187[8]	187	52,173	660	1,000	600/15	550	620	160	490

Contoh perhitungan angker menggunakan K1 yang mempunyai gaya paling besar. Berikut adalah data perencanaan

Unit Kabel = 6-127-115

P = 17967,6 kN

ØA1 = 550 mm

C1 = 845 mm

Ap’ = C1 x C1

= 714025 mm^2

Ap1 = Ap’ – ¼ x π x ØA1^2

$= 714025 - \frac{1}{4} \times \pi \times 550^2$

$= 476562{,}5 \text{ mm}^2$

H $= 900$ mm

Ap2 $= 900 \times 900$

$= 810000 \text{ mm}^2$

f'c $= 70$ MPa

$f_{cp} = 0{,}8.f'c \sqrt{\frac{A2}{A1} - 0{,}2}$

$= 0{,}8 \;.\; 70 \sqrt{\frac{810000}{476562{,}5} - 0{,}2}$

$= 68{,}578$ MPa

Tegangan dibawah pelat angker

Ft $= \frac{17967600}{476562{,}5}$

$= 37{,}703$ MPa $< f_{cp}$ **(memenuhi)**

Untuk perhitungan lainnya akan ditabelkan pada tabel 4.37.

Tabel 4.37 Kontrol tegangan beton angkur

Kabel	Unit	$Ø_{A1}$ (mm)	C_1 (mm)	P (kN)	Teg. Izin pelat (MPa)		
					fcp	ft	Ket
1	6-127-115	550	845	17967,6	68,640	37,762	OK
2	6-109-107	495	775	16717,68	74,858	41,008	OK
3	6-85-84	430	685	13124,16	84,983	40,555	OK
4	6-55-52	380	550	8124,48	113,261	43,036	OK
5	6-19-16	230	335	2499,84	188,068	35,425	OK
6	6-61-59	380	585	9218,16	102,416	40,340	OK
7	6-73-64	430	650	9999,36	92,448	36,113	OK
8	6-85-81	430	685	12655,44	84,983	39,106	OK
9	6-109-103	495	775	16092,72	74,858	39,475	OK
10	6-127-114	550	845	17811,36	68,640	37,434	OK

4.5.4 Kebuuhan Tulangan Pencar

T_{pencar} = $0{,}25.T.\left(1-\frac{C1}{H}\right)$
= $0{,}25.17967{,}6\left(1-\frac{845}{900}\right)$
= 274,505 kN

d_{pencar} = 0,5 (h-2e)
= 0,5 (900-2.0)
= 450 mm

Digunakan sengkang penutup D16 dengan As = 200,96 mm^2

As perlu = T_{pencar} / fy
= 274505 / 400
= 686,26 mm^2

n = As perlu / As pakai
= 686,26 / 200,96
= 3,41 ~ 4 sengkang dekat ujung angkur

Spasi antar sengkang

s = d_{pencar} / (n-1)
= 450 / 3
= 150 mm

Maka dipasang 3D16-150.

Untuk mencegah pecah (*spalling*), dipasang tulangan dengan kuat tarik 2% dari T.

2%.T = 2%.17967,6
= 359,352 kN

As perlu = 2%.T/fy
= 359352/400
= 898,38 mm^2

Digunakan tulangan D16 dengan As = 200,96 mm^2

n = As perlu / As pakai
= 898,38 / 200,96
= 4,47 ~ 5 buah tulangan

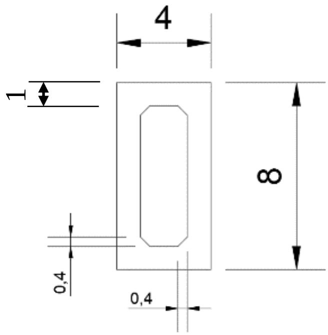
4.6 Perencanaan Pylon

Struktur *pylon* berfungsi memikul beban yang terjadi pada lantai kendaraan, baik berupa beban hidup maupun beban mati, beban dari lantai kendaraan disalurkan melalui kabel ke *pylon* untuk kemudian ditransfer ke pondasi.

4.6.1 Analisa Penampang Pylon

Berikut adalah propertis dari penampang kolom yang digunakan. Untuk lebih jelasnya dapat melihat gambar 4.33.

- Mutu beton (f'c) = 50 MPa
- Mutu tulangan (fy) = 400 MPa



Gambar 4.33 Penampang kolom

Luas = 20,32 m^2
Ix = 137,299 m^4
Iy = 38,91 m^4
rx = 2,599 m
ry = 1,384 m

4.6.2 Penulangan Lentur

1. <u>Arah melintang (*braced frame*)</u>
   - Cek pengaruh kelangsingan SNI 2847-2013 pasal 10.10.1

$$\frac{k.l}{ry} \leq 34 - 12\left(\frac{M1}{M2}\right) \leq 40$$

$\frac{1.25}{1,384} \leq$ 34 - 12$\left(\frac{104021,6}{180480,57}\right)$ $\leq$ 40

18,06 $\leq$ 27,084 $\leq$ 40 (Penampang tidak langsing)

dimana :

k	= faktor panjang efektif komponen struktur = 1
l	= panjang bebas komponen tekan
ry	= jari-jari girasi arah melintang
M1	= momen ujung terfaktor yang lebih kecil
M2	= momen ujung terfaktor yang lebih besar

- Perbesar momen SNI 2847-2013 pasal 10.10.5.2

$$Q = \frac{\sum Pu.\Delta\alpha}{Vu.lc} \leq 0,05$$

$$= \frac{46662,63.0,015}{19146.25} \leq 0,05$$

$$= 0,00146 \leq 0,05 \text{ (kolom tak bergoyang)}$$

dimana :

Pu	= gaya tekan terfaktor (kN)
$\Delta\alpha$	= simpangan relatif (m)
Vu	= gaya geser terfaktor (kN)
lc	= panjang tekan (m)

Prosedur perbesaran momen tak bergoyang SNI 2847-2013 pasal 10.10.16.

$$EI = \frac{0,4.Ec.I}{1+\beta d}$$

$$= \frac{0,4.39323021.38,91}{1+0,5}$$

$$= 408015668,5 \text{ kN.m}^2$$

$$P_c = \frac{\pi^2.EI}{(k.l)^2}$$

$$= \frac{\pi^2.408015668,5}{(1.25)^2}$$

$$= 6436594,056 \text{ kN}$$

Faktor pembesaran momen

$$\delta_{ns} = \frac{Cm}{1-\frac{Pu}{0,75Pc}} \geq 1$$

$$= \frac{1}{1 - \frac{46662,63}{0,75.6436594,056}} \geq 1$$

$= 1,00976 \ \geq 1$

Cm diambil sebesar 1 untuk komponen dengan beban transversal diantara tumpuannya.

Momen desain

Mc $= \delta_{ns}$.Mu

$= 1,00976 \ . \ 180480,57$

$= 182242,143$ kN.m

2. Arah memanjang (*unbraced frame*)
   - Cek pengaruh kelangsingan SNI 2847-2013 pasal 10.10.1

$\frac{k.l}{rx} \leq 22$

$\frac{2.25}{2,599} \leq 22$

19,24 $\leq$ 22 (Penampang tidak langsing)

dimana :

k = faktor panjang efektif komponen struktur = 2

l = panjang bebas komponen tekan

rx = jari-jari girasi arah memanjang

- Perbesar momen SNI 2847-2013 pasal 10.10.5.2

Q $= \frac{\sum Pu.\Delta\alpha}{Vu.lc} \leq 0,05$

$= \frac{46662,63.0,025}{19338,74.25} \leq 0,05$

$= 0,00241 \leq 0,05$ (kolom tak bergoyang)

dimana :

Pu = gaya tekan terfaktor (kN)

$\Delta\alpha$ = simpangan relatif (m)

Vu = gaya geser terfaktor (kN)

lc = panjang tekan (m)

Prosedur perbesaran momen tak bergoyang SNI 2847-2013 pasal 10.10.16.

$$EI = \frac{0{,}4.Ec.I}{1+\beta d}$$
$$= \frac{0{,}4.39323021.137{,}299}{1+0{,}5}$$
$$= 1439736389 \text{ kN.m}^2$$

$$P_c = \frac{\pi^2.EI}{(k.l)^2}$$
$$= \frac{\pi^2.1439736389}{(2.25)^2}$$
$$= 5678089{,}962 \text{ kN}$$

Faktor pembesaran momen

$$\delta_{ns} = \frac{Cm}{1-\frac{Pu}{0{,}75Pc}} \geq 1$$
$$= \frac{1}{1-\frac{46662{,}63}{0{,}75.5678089{,}962}} \geq 1$$
$$= 1{,}011 \geq 1$$

Cm diambil sebesar 1 untuk komponen dengan beban transversal diantara tumpuannya.

Momen desain

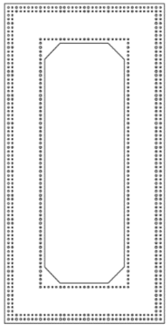
$$Mc = \delta_{ns}.\text{Mu}$$
$$= 1{,}011 \,.\, 727872{,}46$$
$$= 735879{,}057 \text{ kN.m}$$

Untuk menghitung menghitung kemampuan *pylon* digunakan program bantu spColumn dengan memasukan beban yang terjadi sebagai berikut yang dapat dilihat pada tabel 4.38.

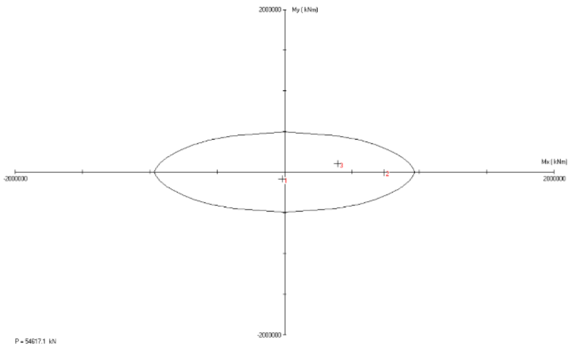
Tabel 4.38 Kombinasi Untuk Pylon

Kombo	Aksial (kN)	Momen-y (kN.m)	Momen-z (kN.m)
Kuat I	-69405.8	-18719.2	-82638.4
Ekstream 1	-60123.4	735879.1	-2241.91
Ekstream 2	-54617.1	393024.3	105036.9

Direncan tulangan longitudinal menggunakan D36 dan tulangan tranversal D22. Kemudian kebutuhan tulangan akan direncanakan dan dianalisa menggunakan spColumn. Untuk lebih jelasnya dapat melihat gambar 4.34 dan 4.35.



Gambar 4.34 Rencana tulangan longitudinal pylon



Gambar 4.35 Diagram Mx dan My pylon

Hasil analisa pylon pada program bantu spColumn dengan jumlah tulangan 626 D36 (3,14%) dengan As = 637190 mm^2. Dengan jarak tulangan sebesar 100 mm.

4.6.3 Penulangan Geser

1. <u>Arah melintang</u>
   Nu = 53289,6 kN

Vu = 19338,74 kN
Ag = 20,32 m^2
Beff = 2x1000 mm
d = 4000-2.100
= 3800 mm
λ = 1
f'c = 50 MPa
Kuat geser beton SNI 2847-2013 pasal 11.2.1.2

$$Vc = 0{,}17\left(1 + \frac{Nu}{14Ag}\right) \lambda.\sqrt{f'c}\ \text{Beff} . d$$

$$= 0{,}17\left(1 + \frac{53289600}{14.20320000}\right) 1.\sqrt{50}\ 2000.\ 3800$$

= 10847167,24 N
= 10847,167 kN

φVc > Vu
0,85 . 10847,167 > 19338,74
9220,092 kN > 19338,74 kN (perlu tulangan geser)

Vu $\leq \phi$Vn
Vu $\leq \phi$Vc + ϕVs
Vs = Vu/ϕ – Vc
= 19338,74/0,85 – 10847,167
= 11904,29 kN

Digunakan sengkang 6 kaki
Av = 6.¼.π.D^2
= 6.¼.π.22^2
= 2279,64 mm^2

$$s = \frac{Av.fy.d}{Vs}$$

$$= \frac{2279{,}64.400.3800}{11904290}$$

= 291,076

Syarat spasi sengkang SNI 2847-2013 pasal 21.3.4.2
- 8D lentur = 8.36 = 288 mm
- 24D sengkang = 24.22 =528 mm
- ½ dimensi kolom terkecil = 4000/2 = 2000 mm

- 300 mm = 300 mm

Digunakan tulangan tranversal arah melintang 6D22-250

2. Arah memanjang

Nu = 60123,4 kN

Vu = 19146 kN

Ag = 20,32 m^2

Beff = 2x1000 mm

d = 8000-2.100

= 7800 mm

λ = 1

f'c = 50 MPa

Kuat geser beton SNI 2847-2013 pasal 11.2.1.2

$$Vc = 0{,}17\left(1 + \frac{Nu}{14Ag}\right) \lambda.\sqrt{f'c}\ \text{Beff} . d$$

$$= 0{,}17\left(1 + \frac{60123400}{14.20320000}\right) 1.\sqrt{50}\ 2000.\ 7800$$

= 22715711,31 N

= 22715,71 kN

φVc > Vu

0,85 . 22715,71 > 19146

19308,35 kN > 19146 kN (gunakan sengkang praktis)

Digunakan tulangan tranversal arah memanjang 6D22-250.

4.6.4 Penulangan Torsi

Desain penulangan torsi berdasarkan SNI 2847-2013 pasal 11.5. Dengan nilau Tu sebesar 73201,62 kN.m

$$Tu \leq \varphi.0{,}083.\lambda.\sqrt{f'c}\left(\frac{Acp^2}{Pcp}\right)$$

$$\leq 0{,}85.0{,}083.1\sqrt{50}\left(\frac{20320000^2}{24000}\right)$$

≤ 5480446758 N.mm

≤ 5480,447 kN.m

dimana :

Acp = luas yang dibatasi oleh keliling luar penampang beton

Pcp = keliling luar penampang beton

λ = faktor modifikasi properti mekanis dari beton

Berdasarkan perhitungan diatas diperlukannya tulangan torsi sebagai berikut.

➢ Tulangan torsi longitudinal

Tn = Tu/ϕ
= 73201,62/0,85
= 86119,553 kN.m

Ao = 0,85.Aoh
= 0,85(13720000)
= 11662000 mm^2

Tn = $\frac{2Ao.At.fyt}{s}.cot\theta$

maka :

$\frac{At}{s} = \frac{Tn}{2Ao.fyt.cot\theta}$

$= \frac{86119553000}{2.11662000.400.cot45}$

= 9,231

Tulangan longitudinal tambahan untuk menahan puntir

$A_l = \frac{At}{s}.Ph.\frac{fyt}{fy}.cot^2\theta$

$= 9,231.20000.\frac{400}{400}.cot^2\theta$

= 184615,75 mm^2

Dipakai D36, As = 1017,36 mm^2

n = A_l/As
= 184615,75/1017,36
= 181,466

Digunakan tulangan torsi longitudinal 200D36.

➢ Tulangan torsi tranversal

Syarat spasi tulangan torsi SNI 2847-2013 pasal 11.5.6

s < Ph/8 = 20000/8 = 2500 mm

s < 300 mm

Digunakan spasi tulangan 300 mm

Tn $= \frac{2Ao.At.fyt}{s}.cot\theta$

maka :

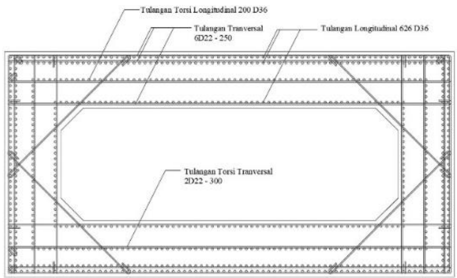
At $= \frac{86119553000.300}{2.211662000.400.cot45}$

$= 152{,}57\ mm^2$

Digunakan 2 kaki D22 As = 2x379,94

Tulangan torsi tanversal digunakan 2D22-300

Berikut adalah sketsa penulangan untuk pylon yang terdapat pada gambar 4.36.



Gambar 4.36 Penulangan Pylon

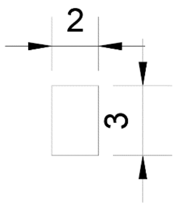
Untuk pylon pada daerah balok penumpu digunakan pylon pejal berdimensi 4m x8m.

4.7 Perencanaan Balok Penumpu

4.7.1 Data Perencanaan Balok

Berikut adalah propertis dari penampang balok yang digunakan. Untuk lebih jelasnya dapat melihat gambar 4.37.

- Mutu beton (f'c) = 50 MPa
- Mutu tulangan (fy) = 400 MPa



Gambar 4.37 Penampang balok

Luas = 6 m^2
Ix = 4,5 m^4
Iy = 2 m^4
rx = 0,886 m
ry = 0,577 m

Berikut adalah gaya dalam yang bekerja hasil analis program bantu MIDAS CIVIL akiat beban ekstream 1.

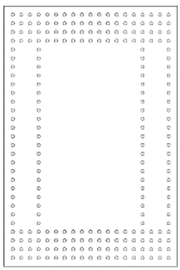
Pu = -16927 kN
Vuz = 21715,86 kN
Vuy = 6144,23 kN
Muy = 96161,32 kN.m
Muz = 27869,81 kN.m
Tu = 240,89 kN.m

Komponen struktur yang mengalami tekan dapat diabaikan bila

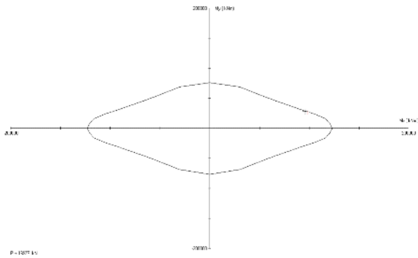
Pu < 0,1.f'c.Ag
12866 < $0{,}1.50.6.10^6$
12866 < 30000 kN (dapat diabaikan)

4.7.2 Penulangan Lentur

Direncanakan tulangan longitudinal menggunakan D36 dan tulangan tranversal menggunakan D22. Kemudian kebutuhan tulangan akan direncanakan dan dianalisa menggunakan spColumn. Untuk lebih jelasnya dapat melihat gambar 4.38 dan 4.39.



Gambar 4.38 Rencana Tulangan Longitudinal Balok



Gambar 4.39 Diagram Mx dan My pylon

Hasil analisa pylon pada program bantu spColumn dengan jumlah tulangan 236 D36 (4%) dengan As = 240219 mm^2. Dengan jarak tulangan sebesar 100 mm.

4.7.3 Penulangan Geser

1. Arah vertikal

Nu = 16927 kN

Vu = 21715,86 kN

Ag = 6 m^2

Beff = 2000 mm

d = 3000-2.100

= 2800 mm

λ = 1

f'c = 50 MPa

Kuat geser beton SNI 2847-2013 pasal 11.2.1.2

$$Vc = 0{,}17\left(1 + \frac{Nu}{14Ag}\right)\lambda.\sqrt{f'c}\ Beff\ .\ d$$

$$= 0{,}17\left(1 + \frac{16927000}{14.6000000}\right)1.\sqrt{50}\ 2000.\ 2800$$

= 8088165,5 N

= 8088,165 kN

φVc > Vu

0,85 . 8088,165 > 21715,86

6874,94 kN > 21715,86 kN (perlu tulangan geser)

Vu ≤ ϕVn

Vu $\leq \phi$Vc + ϕVs
Vs = Vu/ϕ – Vc
= 21715,86/0,85 – 8088,165
= 17459,9 kN
Digunakan sengkang 5 kaki
Av = 5.¼.π.D^2
= 5.¼.π.22^2
= 1899,7 mm^2
s $= \frac{\text{Av.fy.d}}{\text{Vs}}$
$= \frac{1899{,}7\ .400.2800}{17459900}$
= 121,86 mm
Syarat spasi sengkang SNI 2847-2013 pasal 21.3.4.2

- 8D lentur	= 8.36 = 288 mm
- 24D sengkang	= 24.22 =528 mm
- ½ dimensi balok terkecil	= 2000/2 = 1000 mm
- 300 mm	= 300 mm

Digunakan tulangan tranversal arah melintang 5D22-125

2. Arah horizontal
Nu = 16927 kN
Vu = 6144,23 kN
Ag = 6 m^2
Beff = 4000 mm
d = 2000-2.100
= 1800 mm
λ = 1
f'c = 50 MPa
Kuat geser beton SNI 2847-2013 pasal 11.2.1.2
Vc $= 0{,}17\left(1 + \frac{Nu}{14Ag}\right) \lambda.\sqrt{f'c}$ Beff . d
$= 0{,}17\left(1 + \frac{16927000}{14.6000000}\right) 1.\sqrt{50}$ 4000. 1800
= 10399069,92 N
= 10399,07 kN

φVc	> Vu
0,85 . 10399,07	> 6144,23
8839,21 kN	> 6144,23 kN (gunakan sengkang praktis)

Digunakan tulangan tranversal arah memanjang 3D22-300

4.7.4 Penulangan Torsi

Desain penulangan torsi berdasarkan SNI 2847-2013 pasal 11.5. Dengan nilau Tu sebesar 240,89 kN.m

$$Tu \leq \varphi.0{,}083.\lambda.\sqrt{f'c}\left(\frac{Acp^2}{Pcp}\right)$$

$$\leq 0{,}85.0{,}083.1\sqrt{50}\left(\frac{6000000^2}{5000}\right)$$

$$\leq 3591819606 \text{ N.mm}$$

$$\leq 3591{,}82 \text{ kN.m}$$

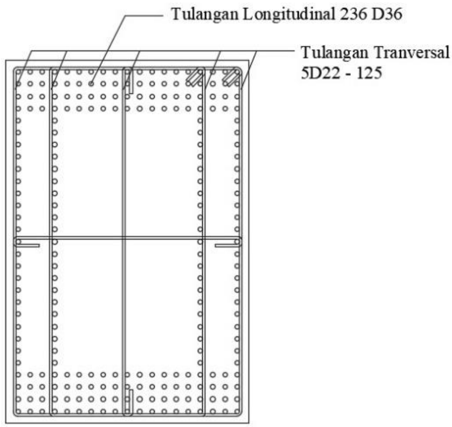
dimana :

Acp = luas yang dibatasi oleh keliling luar penampang beton

Pcp = keliling luar penampang beton

λ = faktor modifikasi properti mekanis dari beton

Berdasarkan perhitungan diatas tidak memerlukan tulangan torsi. Berikut adalah sketsa penulangan untuk balok yang terdapat pada gambar 4.40.



Gambar 4.40 Penulangan balok

4.8 Kontrol Aerodinamis

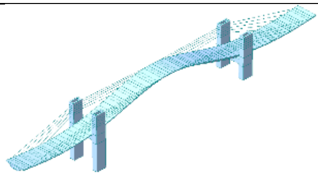
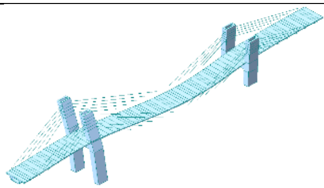
Kontrol terhadap stabilitas aerodinamis pada jembatan *extradosed* perlu dilakukan, karena kegagalan stabilitas aerodinamis merupakan salah satu penyebab terjadinya kegagalan struktur.

Analisa stabilitas pada desain ini meliputi analisa *vortex-shedding* (osilasi gaya akibat pusaran angin) dan *flutter* (efek ayunan).

4.8.1 Frekuensi Alami

Perhitungan frekuensi lentur balok (f_B) dan frekuensi torsi (f_T) menggunakan program bantu MIDAS CIVIL nilai frekuensi alami lentur balok (f_B) dan frekuensi alami torsi (f_T) dapat dicari menggunakan menu *vibration mode shapes*. Berikut adalah beberapa hasil *vibration mode shapes* dari MIDAS CIVIL yang dapat dilihat pada tabel 4.39.

Tabel 4.39 Vibration mode

Mode	Tampak	Frekuensi (Hz)	Keterangan
1		0,987018	Lentur longitudinal
6		2,524294	Torsi Sumbu Y

Berdasarkan hasil analisa MIDAS CIVIL diatas didapatkan nilai f_B pada mode ke-1 dan nilai f_T pada mode ke-5 sebesar :

f_B = 0,987018
f_T = 2,524294

$\frac{f_T}{f_B}$ = 2,557 ≅ 2,5 (Mathivat)

4.8.2 Efek Vortex-shedding

Pada kecepatan angin tertentu yang disebut dengan kecepatan kritis, akan terjadi pusaran angin (*vortex-shedding*). Untuk memperoleh nilai percepatan kritis tersebut, digunakan persamaan angka *Strouhal* (S) sebgai berikut :

S = $\frac{f_B.h}{V}$ (Walther,1999)

0,2 = $\frac{0,987018 \; x \; 3}{V}$

V = 14,805 m/s

dimana :

S = angka *Strouhal* = 0,2

f_B = frekuensi alami lentur balok

h = tinggi lantai kendaraan

V = kecepatan angin yang dihitung berdasarkan angka *Strouhal*

Selanjutnya dilakukan evaluasi efek pusaran dengan angka *Reynold* (Re). Akibat kecepatan angin yang bekerja besarnya anka *Reynold* harus memenuhi persyaratan, yaitu berkisar antara 10^5 – 10^7. Berikut persamaan untuk angka *Reynold.*

Re = $\frac{V.B}{\bar{v}}$

= $\frac{14,805 \; x \; 16}{15.10^{-6}}$

= 15.792.288 **(memenuhi)**

dimana :

Re = angka *Reynold*

V = kecepatan angin yang dihitung berdasarkan angka *Strouhal*

B = lebar lantai kendaraan

$\bar{v}$ = viskositas kinematic udara (0,15 cm^2/s)

Akibat adanya terpaan angin, akan terjadi gaya angkat (*uplift)* yang besarnya dapat dihitung dengan persamaan berikut :

Fo = $\rho \frac{V^2}{2}$C.h

$= 1{,}3 \frac{14{,}805^{\ 2}}{2} 0{,}4 \text{ x } 3$

$= 170{,}97$ Kg

dimana :

Fo = gaya angkat

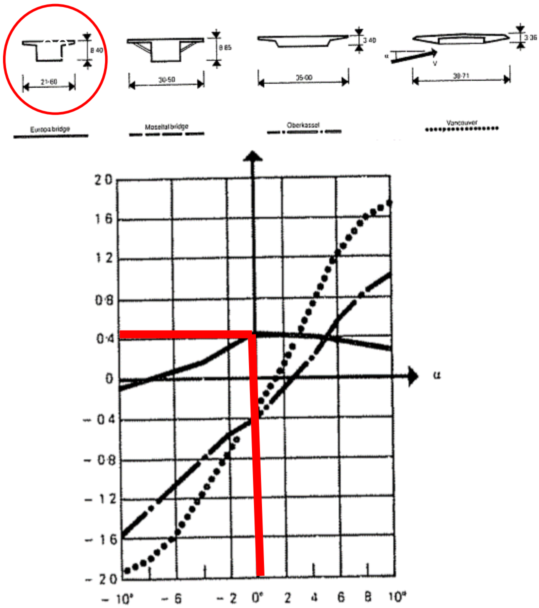
ρ = density/kerapatan udara (1,3 kg/m^3)

V = kecepatan angin yang dihitung berdasarkan angka *Strouhal*

C = koefisien gaya angkat lantai kendaraan (0,4 dari grafik)

h = tinggi lantai kendaraan (m)

Besarnya nilai koefisien C dapat dicari dari grafik berikut ini, lihat gambar 4.41.



Gambar 4.41 Grafik koefisien C_N

Gaya angkat Fo menimbulkan osilasi gelagar yang amplitudonya dapat dihitung menggunakan persamaan berikut :

v $= \frac{\pi . F_o}{\delta . m} v_{maks}$

$= \frac{\pi x 170,97}{0,05 x 45527} 0,01523$

$= 3,591$ mm

dimana :

v = amplitude osilasi

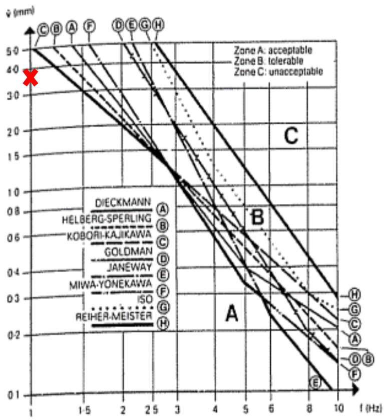
Fo = gaya angkat (Kg)

δ = koefisien peredaman (0,05)

m = berat sendiri lantai per meter panjang (Kg)

v_{maks} = deformasi statis maksimum akbiat berat sendiri (m)

Kontrol keamanan amplitude struktur akibat osilasi ditentukan berdasarkan grafik Rene Walther 1999, dengan mengkobinasikan nilai freukensi lentur dan nilai amplitude akibat osilasi yang tertera pada gambar 4.42.



Gambar 4.42 Kontrol keamanan amplitude

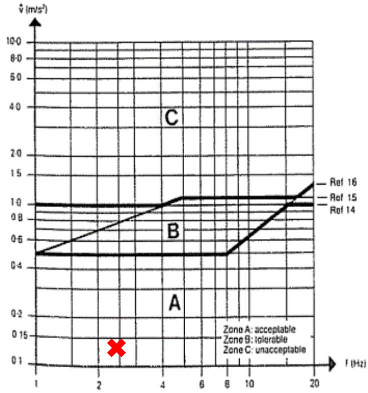
Dapat dilihat pada gambar diatas bahwa nilai hasil plot amplitude dan frekuensi lentur berada pada zona aman. Bila perlu, perhitungan dapat dilanjutkan dengan mencari nilai percepatan getaran yang didapatkan dari persamaan berikut :

$\ddot{v} = 4\pi^2.f_B^2.\hat{v}$

$= 4x\pi^2 x0,987018^2 x0,003591$

$= 0,138$ m/s

Kontrol keamanan percepatan akibat osilasi ditentukan berdasarkan grafik Rene Walther 1999 dengan mengkobinasikan nilai torsi f_T dengan nilai percepatan akibat osilasi yang tertera pada gambar 4.43.



Gambar 4.43 Kontrol keamanan percepatan

Dapat dilihat pada gambar diatas bahwa nilai plot antara percepatan akibat osilasi dan nilai frekuensi torsi berada dalam zona aman.

4.8.3 Efek *Flutter* (Ayunan)

Hasil akhir perhitungan *flutter* adalah didapatkannya nilai kecepatan kritis actual struktur ($V_{kritis.aktual}$). Ababila nilai kritis actual struktur lebih besar dari nilai kecepatan angin rencana, maka struktur dinyatakan aman .

Untuk mendapatkan kecepatan kritis teoritis ($V_{kritis.teoritis}$), dapat digunakan metode Perhitungan *flutter* menggunakan metode *Kloppel* sebagai berikut :

$V_{kritis.teoritis}$ = $2.\pi.f_B.b$

dimana b = ½ lebar lantai kendaraan

Untuk mencari nilai $V_{kritis.teoritis}$ dapat dicari secara grafis dari gambar 4.47 dan tergantung dari 3 persamaan berikut ini :

1. $\mu = \frac{m}{\pi.\rho.b^2}$

$= \frac{45527}{\pi x 1{,}3 x 8^2} = 174{,}267$

2. r/b = 1,647/8 = 0,205875
3. f_T/f_B = 2,557

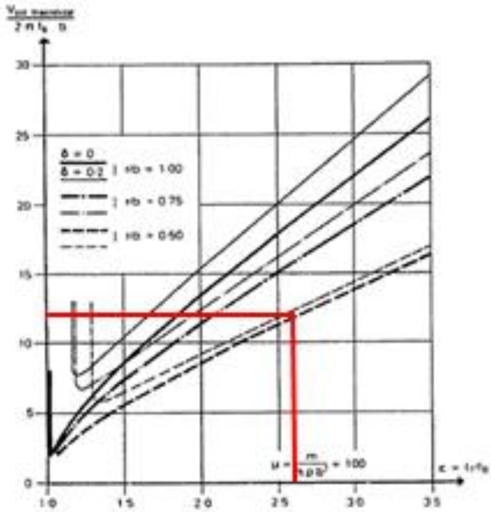
dimana :

m = berat sendiri lantai per meter panjang (Kg)

ρ = density/kerapatan udara (1,3 kg/m^3)

b = ½ lebar lantai kendaraan

r = radius girasi penampang (m)



Gambar 4.44 Grafik Vkritis.Teoritis

Dari grafik diatas didapatkan nilai :

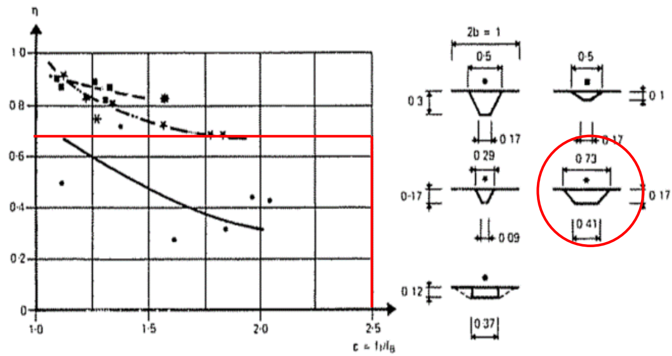
$\frac{V_{kritis.teoritis}}{2.\pi.f_B.b}$ = 12

$V_{kritis.teoritis}$ = 12x(2.π.$f_{B.}$b)

= 12x(2xπx0,987018x8)

= 595,053 m/s

Besar kecepatan krtis teoritis ini harus dikoreksi menjadi kecepatan kritis actual, menggunakan grafik pada gambar 4.45 berikut :



Gambar 4.45 Grafik Koreksi Kecepatan Kritis

Dari grafik diatas, dengan menyesuaikan bentuk penampang yang paling mendekati didapatkan nilai η = 0,67.

$V_{kritis.aktual}$ = 0,67 x 595,053
= 398,68 m/s

Akan tetapi pada kondisi nyata, angin tidak selalu mengenai lantai kendaraan dalam arah horizontal secara sempurna. Terkadang nilai α dapat berubah berkisar antara 6^o. Sebagai pembanding dilakukan koreksi kecepatan kritis actual dengan kemiringan sudut angin sebesar 6^0 sebagai berikut :

$V_{kritis.aktual}\ 6^o$ = 1/3 x $V_{kritis.aktual.}$
= 132,89 m/s

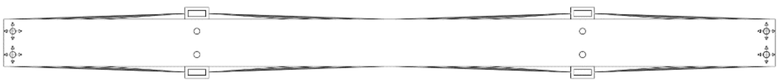
Sedangkan untuk kecepatan angin rencana pada jembatan *extradosed* ini sebesar 108,723 km/jam atau setara 30,2 m/s. Sehingga analisa efek *flutter* ini memenuhi syarat yang telah disyaratkan yaitu $V_{kritis.aktual} > V_{rencana}$. Analisa ini perlu dilanjutkan dengan pembuktian menggunakan jembatan model berskala pada *wind tunnel*, agar diperoleh hasil yang lebih akurat.

4.9 Perencanaan Perletakan

Pada perencanaan jembatan *extradosed* di Widang ini akan digunakan landasan jembatan yang berfungsi meneruskan beban dari bangunan atas ke bangunan bawah jembatan. Landasan yang digunakan pada jembatan ini merupakan pelat baja sirkular (pot bearing). Dengan penggunaan landasan pot bearing tersebut akan mampu mengakomodasi perpindahan arah akibat beban yang ada. Desain dari pot bearing merupakan elemen fabrikasi dari *TOBE FR4 Pot Bearings*.

4.9.1 Layout Penempatan Perletakan Jembatan

Perencanaan jembatan *extradosed* di Widang ini merupakan jembatan bentang panjang dimana besarnya pergerakan cukup mempengaruhi stabilitas jembatan. Oleh karena itu hal ini dapat diminimalkan dengan pengaturan penempatan posisi landasan. Untuk lebih jelasnya dapat melihat gambar 4.46.



Gambar 4.46 Layout perletakan jembatan

dimana :

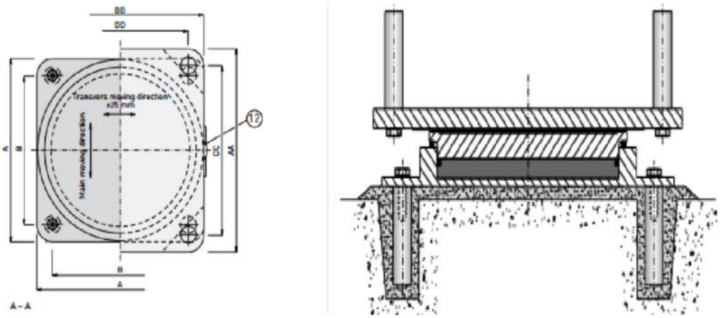
 = pot bearing tipe multidirectional

 = pot bearing tipe unidirectional

 = pot bearing tipe fixed

4.9.2 Kontrol Desain Perletakan Pot Bearing

- Multidirectional pot bearing type A
  Jenis perletakan ini akan memungkinkan 2 gerakan arah lateral dan longitudinal untuk lebih jelasnya dapat melihat gambar 4.47.



Gambar 4.47 Type perletakan multidirectional

Dari hasil analisa MIDAS CIVIL didapatkan reaksi dan pergerakan pada perletakan untuk pot bearing arah x dan arah y akibat beban extream 1.

- Translasi X = 0,033 m
- Translasi Y = 0,002 m
- V = 9715,7/2
  = 4857,85 N

Dari hasil reaksi vertikal pada perletakan pot bearing, maka akan direncanakan multidirectional pot bearing tipe 50. Dimensi dan spesifikasi dapat dilihat pada gambar 4.48 dan tabel 4.40 berikut.

Multidirectional pot bearing Type A

Movement	Primary	Secondary
	± 50 mm	±20 mm
	±100 mm	±20 mm

Gambar 4.48 Pergerakan Pot Bearing Tipe A

Tabel 4.40 Dimensi Multidirectional Pot Bearing

Load table for multidirectional pot bearings at ULS

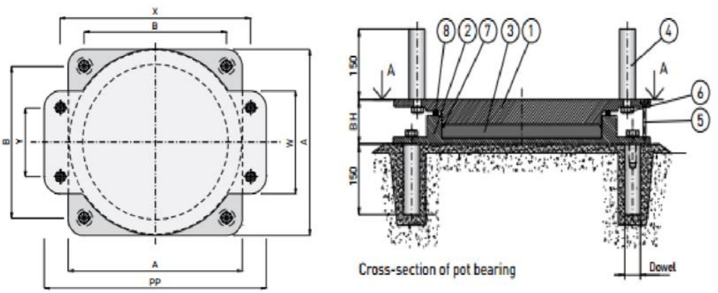
Type		10	20	30	40	50	60	70	80	90	100	110	120	130
Vmax	kN	520	1590	3040	4690	6090	7670	9060	10570	13480	16250	21450	27400	33400

Dimension table for multidirectional pot bearings (all measurements in mm)

A	mm	200	295	390	470	535	590	640	685	770	840	955	1070	1170
B	mm	149	232	315	386	440	470	510	540	600	650	730	810	880
AA ± 50	mm	300	420	495	560	635	675	710	745	805	855	945	1025	1105
CC ± 50	mm	249	357	420	476	540	580	615	650	710	760	850	930	1010
AA ± 100	mm	400	520	595	660	735	775	810	845	905	955	1045	1125	1205
CC ± 100	mm	349	457	520	576	640	680	715	750	810	860	950	1030	1110
BB	mm	200	300	380	450	500	550	590	630	700	760	860	960	1050
DD	mm	149	237	305	366	405	455	495	535	605	665	765	865	955
BH	mm	72	78	88	97	102	111	114	118	129	139	151	165	177
Ø	mm	35	50	50	50	50	50	50	50	50	50	50	50	50
Bolt dim.	mm	M16	M20	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24
A_{PTFE}	mm^2	14314	33979	57256	98980	132025	173494	204282	237583	301907	363168	477836	608212	738981
Ao	mm^2	27964	59634	89488	140232	179113	226949	261981	299526	371278	438905	564184	705171	845490
Au	mm^2	20503	49706	86345	126652	162621	200323	236287	275218	347768	417267	544134	692919	837679
Weight ± 50	kg	25	60	94	134	170	213	248	287	376	468	636	847	1103
Weight ± 100	kg	28	65	100	141	178	221	257	297	387	480	649	862	1119

- Fixed pot bearing

  Jenis perletakan ini hanya tidak memungkinkan adanya gerakan arah lateral. Dipakai tipe F dapat dilihat pada gambar 4.49.



Gambar 4.49 Tipe perletakan fixed

Dari hasil analisa MIDAS CIVIL didapatkan reaksi dan pergerakan pada perletakan untuk pot bearing arah x dan y akibat beban ekstream 1.

- Translasi X = 0 m
- Translasi Y = 0 m
- V = 35873,36/2
  = 17936,68 kN
- H = 4067,25/2
  = 2033,625 kN

Akan direncanakan fixed pot bearing tipe 110. Dimensi dan spesifikasi dapar dilihat seperti pada tabel 4.41 dibawah.

Tabel 4.41 Dimensi fixed pot bearing

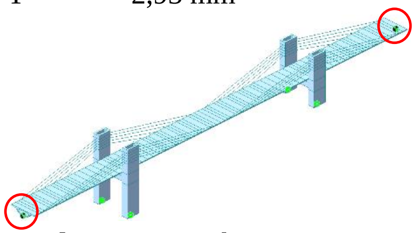
Load table for fixed pot bearings at ULS

Type		10	20	30	40	50	60	70	80	90	100	110	120	130
V_{max}	kN	520	1590	3040	4690	6090	7670	9060	10570	13480	16250	21450	27400	33400
V_{min}	kN	60	380	1210	2070	2790	3400	4070	4670	5670	6830	8110	9830	11360
H_{max}	kN	220	420	720	950	1130	1280	1450	1600	1850	2140	2460	2890	3260
Dimensions fixed bearing (all measurements in mm)														
A	mm	200	295	390	470	535	590	640	685	770	840	955	1070	1170
B	mm	149	232	315	386	440	470	510	540	600	650	730	810	880
W	mm	115	165	200	250	300	300	350	375	425	450	500	550	600
X	mm	230	335	440	520	595	655	700	745	830	910	1015	1135	1230
Y	mm	64	102	125	166	205	205	255	280	330	355	405	455	505
BH	mm	56	56	67	72	81	90	93	101	108	118	130	145	161
PP	mm	281	398	515	604	690	750	795	840	925	1005	1110	1230	1325
Ø	mm	35	50	50	50	50	50	50	50	50	50	50	50	50
Bolt dim.		M16	M20	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24
h	mm	29,5	29,9	36,2	42,2	47,6	52,0	55,5	58,8	66,0	71,2	79,2	88,8	97,0
Rubber D	mm	120	210	290	360	410	460	500	540	610	670	770	870	960
Ao	mm^2	17671	45239	80425	119459	152053	188574	220618	255176	321699	384845	502655	636173	769769
Au	mm^2	20503	49706	86345	126652	162621	200323	236287	275218	347768	417267	544134	692919	837679
Weight	kg	20	43	70	96	129	164	193	236	301	384	529	722	955

4.10 Perencanaan Expantion Joint

Pada perencanaan expantion joint ini direncanakan pada gelagar box girder untuk menahan gaya aksial vertikal, horizontal x dan y . Tipe expantion joint adalah modular expantion joint. Yang dimana modular ini adalah gabunngan dari dua atau lebih *Strip Seal Joint* untuk mengakomodasi movement yang sangat besar. Modular Joint dibuat untuk mengakomodasi movement lebih dari 100 mm. Besarnya modular joint tergantung besarnya movement. Modular joint dirancang untuk jembatan dengan bentang yang panjang dengan kemampuan movement sampai 2 m. Biasanya modular joint digunakan untuk movement antara 150 mm sampai 600 mm. Ada 3 bagian utama dari joint ini, yaitu: sealer, separator beam, dan support bar (Transportation Research Board, 2003). Berikut adalah data perencanaan menggunakan kombinasi beban extream 1.

- Deformasi X = 33 mm
- Deformasi Y = 2,93 mm



Gambar 4.50 Letak expantion joint

Berikut adalah data spesifikasi expantion joint, untuk lebih jelasnya dapat melihat tabel 4.42.

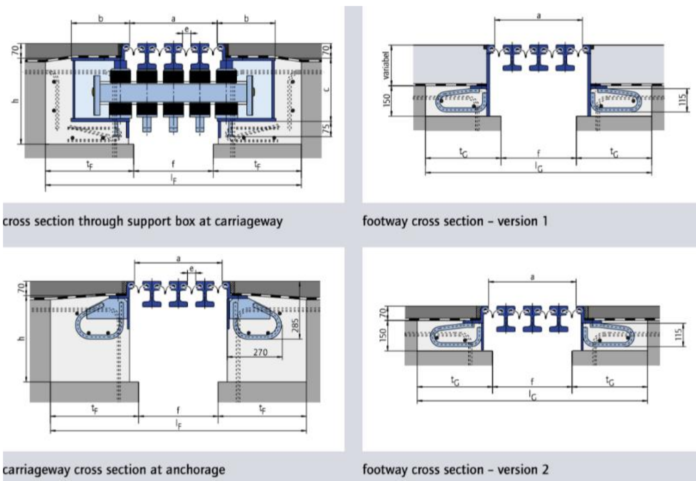
Tabel 4.42 Tipe Expantion Joint

n	type	u_x [mm]	u_y *) [mm]	u_z *) [mm] mid-position	α [°]	Δα	β [°]
2	DS160	130 (160)	± 80	± 10			
3	DS240	195 (240)	± 120	± 15			
4	DS320	260 (320)	± 160	± 20			
5	DS400	325 (400)	± 200	± 25			
6	DS480	390 (480)	± 240	± 30			
7	DS560	455 (560)	± 280	± 35			
8	DS640	520 (640)	± 320	± 40	90° ± 45°	any	any
9	DS720	585 (720)	± 360	± 40			
10	DS800	650 (800)	± 400	± 40			
11	DS880	715 (880)	± 440	± 40			
12	DS960	780 (960)	± 480	± 45			
13	DS1040	845 (1040)	± 520	± 45			
14	DS1120	910 (1120)	± 560	± 45			
15	DS1200	975 (1200)	± 600	± 45			

Dari data spesifikasi maka expantion joint yang digunakan adalah tipe DS160. Berikut adalah dimensi dari expantion joint yang tertera pada tabel 4.43 dan gambar 4.50.

Tabel 4.43 Dimensi expantion joint

Presetting of gap dimension e = 30 mm											
MAURER exp. joint		design data			concrete-recess dimensions			concrete-gap dimensions			
n	type	a [mm]	b [mm]	c [mm]	h [mm]	t_F [mm]	t_G [mm]	f_{min} [mm]	f_{max} [mm]	l_F [mm]	l_G [mm]
2	DS160	150	215	290	420	350	350	115	130	815	815
3	DS240	270	255	300	430	395	380	225	250	1015	985
4	DS320	390	285	310	440	435	390	300	370	1170	1080
5	DS400	510	355	320	450	510	400	410	490	1430	1210
6	DS480	630	380	330	460	550	410	520	610	1620	1340
7	DS560	750	410	340	470	590	420	630	730	1810	1470
8	DS640	870	430	350	480	620	430	740	850	1980	1600
9	DS720	990	460	360	490	660	440	850	970	2170	1730
10	DS800	1110	490	370	500	690	450	960	1090	2340	1860
11	DS880	1230	515	380	510	730	460	1070	1210	2530	1990
12	DS960	1350	550	390	520	770	470	1180	1330	2720	2120
13	DS1040	1470	585	400	530	820	480	1290	1450	2930	2250
14	DS1120	1590	615	410	540	860	490	1400	1570	3120	2380
15	DS1200	1710	645	420	550	900	500	1510	1690	3310	2510



Gambar 4.51 Dimensi expantion joint

BAB V
PENUTUP

5.1 Kesimpulan

Dari hasil perencanaan, analisa, dan perhitungan struktur yang telah dilakukan pada jembatan *Extradosed* ini, maka didapatkan kesimplan sebagai berikut :

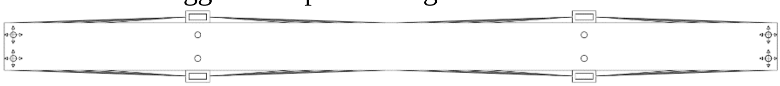
1. Panjang total dari desain jembatan ini sebesar 260 m yang meliputi 130 m bentang utama dan bentang tepi 2@65 m. dengan lebar jembatan sebesar 16 m. Sedangkan untuk susunan kabel berupa *semi-harp pattern*.
2. Tiang sandaran jembatan terbuat dari profil baja *rectangular hollow* 200x200x8 setinggi 1,7 m. Dan pipa horizontal menggunakan profil *circular hollow* Ø3,5" sepanjang 2 m.
3. Pelat lantai kendaraan berupa tapered box girder dengan ketinggian pada pylon sebesar 4,5 m dan pada tengah bentang sebesar 3 m.
4. Analisa metode pelaksanaan menggunakan program bantu MIDAS CIVIL berupa *backward solution*. Sedangkan saat pelaksanaan konstruksi menggunakan *form traveler*.
5. Tendon konstruksi pada box menggunakan VSL 6-31-31, sedangkan untuk tendon menerus menggunakan VSL 6-55-55.
6. Tulangan susut memanjang pada box menggunakan D22-300, sedangkan ke arah melintang sebagai berikut :

	Tul +	Tul -
Flens atas	2D32 - 100	D32 - 150
Web	2D32 - 100	D32 - 150
Flens bawah	D32 - 300	D32 - 150

7. Penulangan blok angkur
   a. Tulangan *bursting* 3D16
   b. Tulangan *spalling* 5D16
8. Berdasarkan beban-beban yang bekerja didapatkan kebutuhan kabel jembatan *extradosed* sebagai berikut :

Kabel	Unit	Kabel	Unit
1	6-127-115	6	6-61-59
2	6-109-107	7	6-73-64
3	6-85-84	8	6-85-81
4	6-55-52	9	6-109-103
5	6-19-16	10	6-127-114

9. Stabilitas aerodinamis struktur jembatan setelah dihitung menggunakan rumus empiris untuk efek *vortex-shedding* dan pada cek psikologis masih dominan berada pada zona aman. Sedangkan untuk efek ayunan (*flutter*) juga berada pada zona aman. Untuk kecepatan angin kritis memiliki nilai lebih besar dibandingkan dengan kecepatan angin desain yaitu 132,89 m/s dibanding 30,2 m/s.
10. Struktur *pylon* menggunakan box berdimensi 4m x 8m dengan tebal dinding 1m menggunakan struktur beton bertulang. Dengan tulangan longitudinal terpasang 626D36, tulangan torsi 200D36, tulangan tranversal melintang 6D22-250, tulangan tranversal memanjang 6D22-250.
11. Balok penumpu berdimensi 2m x 3m berupa beton bertulang. Dengan tulangan longitudinal 236D36, tulangan tranversal arah vertikal 5D22-125, tulangan tranversal arah horizontal 3D22-300.
12. Perletakan menggunakan pot bearing



TOBE FR4 Pot Bearings terdapat dua jenis, yaitu :
a. Multidirectional pot bearing tipe 50 (bergerak bebas 2 arah)
b. Fixed pot bearing tipe 110 (tidak bergerak bebas 2 arah)

13. Menggunakan tipe expantion joint tipe modular joint dengan spesifikasi DS160.

5.2 Saran

Hasil pengerjaan laporan tugas akhir ini masih terdapat kekurangan, maka dari itu untuk hasil yang lebih baik perlu adanya beberapa hal yang diperhatikan dalam melakukan desain sejenis untuk kedepannya, antara lain sebagai berikut :

1. Untuk desain yang sebenarnya dalam analisa dinamis akibat beban angin selain dilakukan kontrol menggunakan rumus empiris perlu juga dimodelkan penuh enggunakan terowongan angin. Hal ini dimaksudkan agar ketelitian dalam desain lebih akurat.
2. Permodelan perletakan dan expantion joint pada jembatan ini sebaiknya dimodelkan pada desain sebenarnya agar memperoleh desain yang lebih aman.
3. Dalam kasus jembatan kali ini, alangkah lebih baiknya di tinjau kembali untuk layout jembatan pada daerah sungai. Dengan cara menggeser pylon jembatan ke arah kiri agar kaki pylon satu dengan yang lain memiliki ketinggian yang sama. Dan juga bila hal itu terjadi, dipertimbangkan juga biaya jembatan pendekat pada arah kanan jembatan dan pembongkaran sisi kiri jembatan (apa masih memungkinkan) dibandingkan dengan biaya letak pylon tetap.
4. Bila menggunakan box penuh pada pylon jembatan, perlu ditinjau perlemahan dinding pylon akibat adanya balok penumpu yang cukup besar.

“Halaman ini sengaja dikosongkan”

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LABORATORIUM MEKANIKA TANAH & BATUAN
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Kampus ITS, Keputih Sukolilo Surabaya
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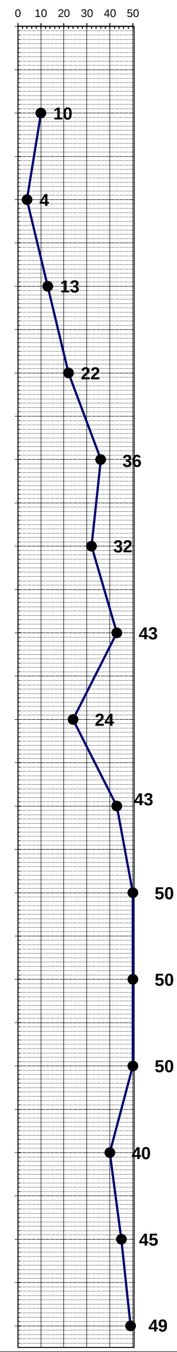
D R I L L I N G L O G

KLIEN	=	**PT. WASIS KARYA NUGRAHA**	**TIPE BOR**	=	**ROTARY DRILLING**	Remarks.
NAMA PROYEK	=	**JEMBATAN BONCONG**	**TANGGAL MULAI**	=	**5 MEI 2015**	UD = Undisturb Sample
TITIK BOR	=	**BH-1 (SISI TIMUR JEMBATAN)**	**TANGGAL SELESAI**	=	**7 MEI 2015**	CS = Core Sample
MUKA AIR TANAH	=	**2,20 M**	**MASTER BOR**	=	**OSIAS TENIS**	SPT = SPT Test
LOKASI PROYEK	=	**DS. BONCONG, KEC. BANDAR - TUBAN**				

Scale in m	Elevation (LWS) in m	Depth in m	Thickness in m	Legend	Type of Soil	Colour	Relative Density or Consistency	General Remarks	UD / CS Depth in m	UD / CS Sample Code	SPT TEST Depth in m	SPT TEST Sample Code	N-Value Blows/30 cm	Blows per each 15 cm: 15 cm	15 cm	15 cm
0.00	0.00				START OF BORING											
1.00	-1.00															
2.00	-2.00				TIMBUNAN LIMESTONE	COKLAT TERANG	VERY LOOSE	SPT 4 s/d 10			-2.00 -2.50	SPT 1	10	2	4	6
3.00	-3.00															
4.00	-4.00										-4.00 -4.50	SPT 2	4	1	1	3
5.00	-5.00															
6.00	-6.00				LEMPUNG BERLANAU	ABU-ABU KECOKLATAN	STIFF TO VERY STIFF	SPT 13 s/d 22	-5.50 -6.00	UD 01	-6.00 -6.50	SPT 3	13	3	5	8
7.00	-7.00															
8.00	-8.00										-8.00 -8.50	SPT 4	22	5	8	14
9.00	-9.00															
10.00	-10.00										-10.00 -10.50	SPT 5	36	7	15	21
11.00	-11.00															
12.00	-12.00								-11.50 -12.00	UD 02	-12.00 -12.50	SPT 6	32	9	8	24
13.00	-13.00															
14.00	-14.00				LANAU LEMPUNG BERPASIR	ABU-ABU TERANG	VERY STIFF TO HARD	SPT 24 s/d 43			-14.00 -14.50	SPT 7	43	7	16	27
15.00	-15.00															
16.00	-16.00										-16.00 -16.50	SPT 8	24	5	9	15
17.00	-17.00															
18.00	-18.00								-17.50 -18.00	UD 03	-18.00 -18.50	SPT 9	43	6	15	28
19.00	-19.00															
20.00	-20.00										-20.00 -20.50	SPT 10	>50	8	17	20/5
21.00	-21.00															
22.00	-22.00				PASIR BERLANAU	ABU-ABU TERANG	DENSE	SPT >50			-22.00 -22.50	SPT 11	>50	5	19	28/10
23.00	-23.00															
24.00	-24.00								-23.50 -24.00	UD 04	-24.00 -24.50	SPT 12	>50	9	16	22/5
25.00	-25.00															
26.00	-26.00										-26.00 -26.50	SPT 13	40	7	16	24
27.00	-27.00															
28.00	-28.00				LANAU LEMPUNG BERPASIR	ABU-ABU TERANG	HARD	SPT 40 s/d 49			-28.00 -28.50	SPT 14	45	6	18	27
29.00	-29.00															
30.00	-30.00										-30.00 -30.50	SPT 15	49	8	19	30

Standard Penetration Test — N - Value



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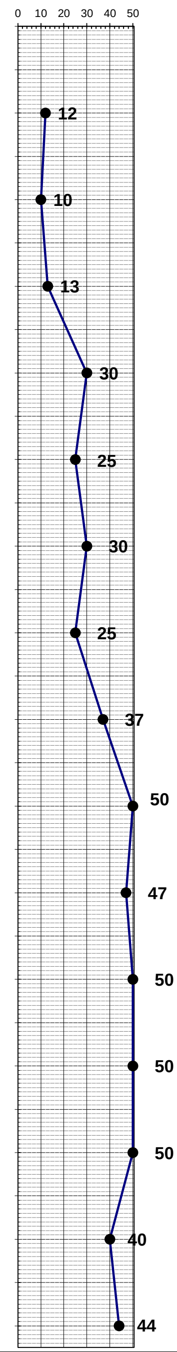
D R I L L I N G L O G

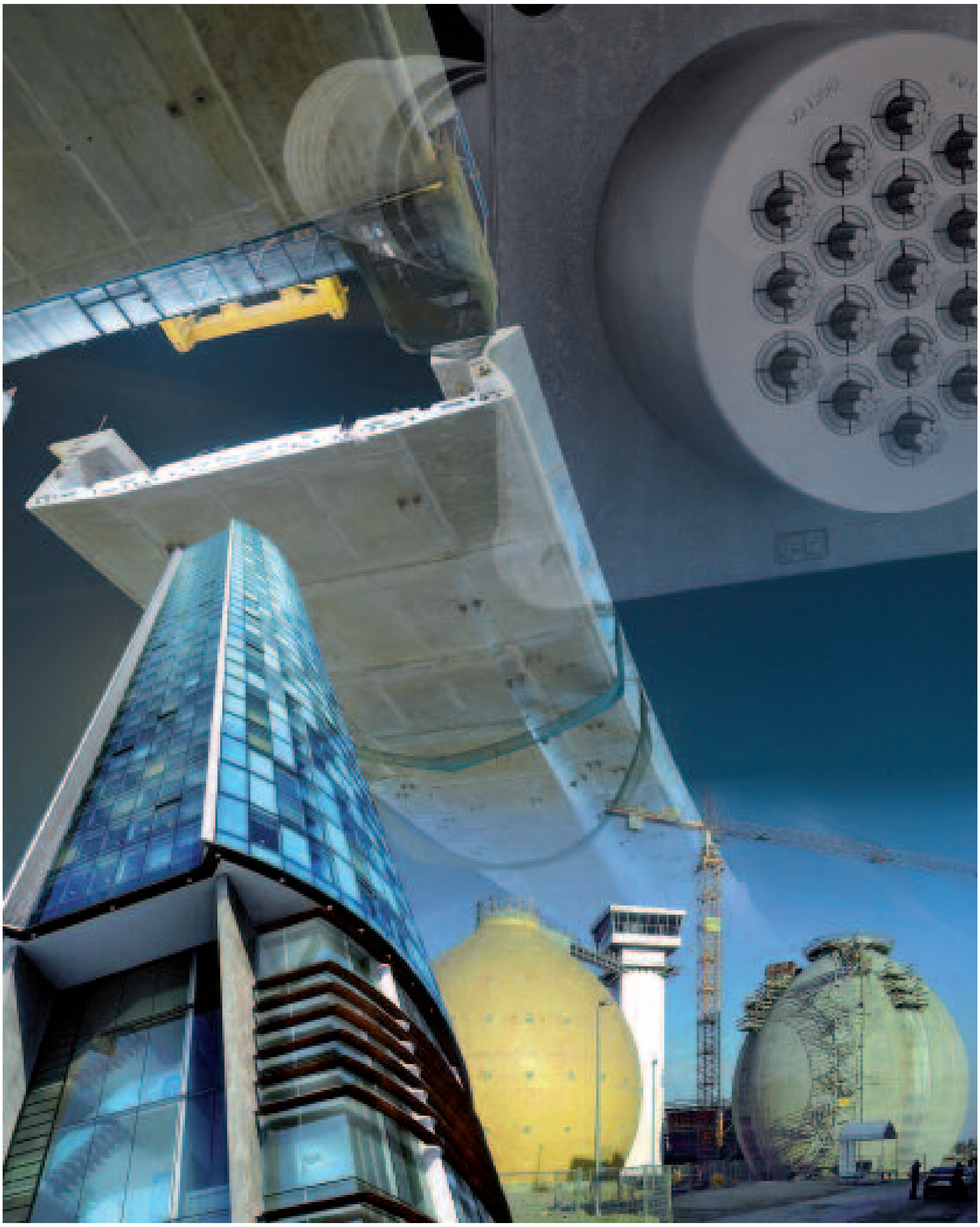
KLIEN	= **PT. WASIS KARYA NUGRAHA**	**TIPE BOR**	= **ROTARY DRILLING**	Remarks.	
NAMA PROYEK	= **JEMBATAN BONCONG**	**TANGGAL MULAI**	= **8 MEI 2015**	UD	= Undisturb Sample
TITIK BOR	= **BH-2 (SISI BARAT JEMBATAN)**	**TANGGAL SELESAI**	= **10 MEI 2015**	CS	= Core Sample
MUKA AIR TANAH	= **3,35 M**	**MASTER BOR**	= **OSIAS TENIS**	SPT	= SPT Test
LOKASI PROYEK	= **DS. BONCONG, KEC. BANDAR - TUBAN**				

Scale in m	Elevation (LWS) in m	Depth in m	Thickness in m	Legend	Type of Soil	Colour	Relative Density or Consistency	General Remarks	UD / CS Depth in m	UD / CS Sample Code	SPT TEST Depth in m	SPT TEST Sample Code	N-Value Blows/30 cm	Blows per each 15 cm: 15 cm	15 cm	15 cm
0.00	0.00	START OF BORING														
1.00	-1.00				TIMBUNAN LIMESTONE	COKLAT TERANG	LOOSE	SPT = 12								
2.00	-2.00										-2.00 -2.50	SPT 1	12	3	5	7
3.00	-3.00															
4.00	-4.00										-4.00 -4.50	SPT 2	10	2	4	6
5.00	-5.00				LEMPUNG	ABU-ABU KECOKLATAN	STIFF	SPT 10 s/d 13								
6.00	-6.00								-5.50 -6.00	UD 01	-6.00 -6.50	SPT 3	13	3	5	8
7.00	-7.00															
8.00	-8.00										-8.00 -8.50	SPT 4	30	6	16	14
9.00	-9.00															
10.00	-10.00										-10.00 -10.50	SPT 5	25	4	9	16
11.00	-11.00				LEMPUNG BERLANAU	ABU-ABU	VERY STIFF	SPT 25 s/d 30								
12.00	-12.00								-11.50 -12.00	UD 02	-12.00 -12.50	SPT 6	30	6	12	18
13.00	-13.00															
14.00	-14.00										-14.00 -14.50	SPT 7	25	5	10	15
15.00	-15.00															
16.00	-16.00										-16.00 -16.50	SPT 8	37	9	15	22
17.00	-17.00															
18.00	-18.00								-17.50 -18.00	UD 03	-18.00 -18.50	SPT 9	>50	8	17	25/5
19.00	-19.00															
20.00	-20.00										-20.00 -20.50	SPT 10	47	9	19	28
21.00	-21.00				LANAU BERPASIR	ABU-ABU	HARD	SPT 47 s/d >50								
22.00	-22.00										-22.00 -22.50	SPT 11	>50	13	22/10	
23.00	-23.00															
24.00	-24.00								-23.50 -24.00	UD 04	-24.00 -24.50	SPT 12	>50	11	17	26
25.00	-25.00															
26.00	-26.00										-26.00 -26.50	SPT 13	>50	11	25/5	
27.00	-27.00															
28.00	-28.00				LANAU LEMPUNG BERPASIR	ABU-ABU	HARD	SPT 40 s/d >50			-28.00 -28.50	SPT 14	40	8	16	24
29.00	-29.00															
30.00	-30.00										-30.00 -30.50	SPT 15	44	8	18	26

Standard Penetration Test — N - Value



VSL POST-TENSIONING SOLUTIONS



CONCEPTUAL DESIGN
ENGINEERING SOLUTIONS
CONSTRUCTION PARTNER
FOR BRIDGES, BUILDINGS
CONTAINMENT
STRUCTURES, SLAB ON
GRADE, SPECIAL
STRUCTURES, REPAIR
AND STRENGTHENING

A REPUTATION FOR EXCELLENCE SINCE

VSL's leadership in post-tensioning

VSL is a recognised leader in the field of special construction methods. Well-proven technical systems and sound in-house engineering are the basis of the group's acknowledged reputation for innovative conceptual designs and engineering solutions, for reliability, quality and efficiency.

VSL executes all works using its own staff and equipment.

Gateway, Australia - 2008

VSL – post-tensioning as the core business

For decades, VSL has designed, manufactured and installed durable, state-of-the art post-tensioning systems complying with international standards and approval guidelines for both new and existing structures. Services and products are all aimed at delivering the optimal solution for the customer.

Deep Bay Link, Hong Kong - 2005

The VSL Network

VSL operates as a multinational group of companies whose subsidiaries and licensees are organised into closely-cooperating regional units. Customers benefit greatly from the continuing development of VSL's special construction methods and from the exchange of information taking place within the VSL Network.

Dubai Festival City, UAE -2006

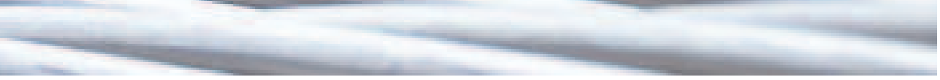
VSL's aim of creating innovative solutions by adapting proven experience is supported by the ability to identify and share immediately the best ideas that have been introduced anywhere within the network. The solutions are developed and tailored for clients worldwide.

VSL Subsidiaries execute all work using their own personnel and equipment: technical consultancy and support during planning and all phases of construction are part of VSL's value-added service, which is tailored to suit the client's needs.

VSL – your construction partner

With offices throughout the world, VSL offers a comprehensive range of professional, high-quality services for all kinds of projects, from feasibility studies and preliminary designs to alternative proposals, contractor consultancy services and field installation. All are aimed at finding the best possible solutions with the best value for money. VSL's involvement seeks to provide fully-customised solutions adapted to the client's requirements. Its worldwide network allows VSL to offer a high degree of competence and flexibility, participating with a spirit of co-operation to find the most appropriate solutions. VSL's goal is to be a privileged partner for engineers and contractors.

CREATING SOLUT



1956

Changing the way we do business

For VSL, sustainable development means striking a balance in its development model between the economic profitability of its businesses and their social and environmental impacts. That commitment is formalised into the VSL sustainable development program which focuses on safety, use of fewer scarce materials and less energy and production of less pollution and waste.

VSL – guided by a strong QSE culture

VSL's leading position is based on a rigorous and committed quality culture. The QSE (quality, safety, environment) policy is VSL's first priority. Local teams ensure co-ordination of actions, encourage sharing of experience and promote best practice, with the aim of continuously improving performance. In VSL's culture, employees are vitally important to the competitiveness and prosperity of the company. VSL is committed to maintaining the highest levels of client satisfaction and personnel safety.

CONTRIBUTING TO SUSTAINABLE SOLUTIONS

Post-tensioning reduces CO_2 emissions by up to 27%

Generally the use of VSL Post-tensioning delivers the maximum cost-benefit for a project as well as having a beneficial impact on its sustainability and CO_2 emissions during construction. Compared with conventional reinforced concrete slabs, the use of post-tensioning results in more durable structures with reduced concrete volumes, lowering the CO_2 emissions by up to 27%.

Post-tensioning offers significant reductions

Materials and quantities	RC (kg CO_2/m^2)	PC (kg CO_2/m^2)
Concrete (300 kg cement / m^3)	105.1	84.0
Reinforcing steel	24.8	8.3
Post-tensioning steel	0	3.0
Total CO_2 emission	129.9	95.3

The overall reduction of CO_2 emission can achieve up to 27%!

RC: Reinforced concrete *PC: Post-tensioned concrete*

Ras Laffan LNG, Qatar - 1996

The VSL Academy

Competence is a key factor and VSL adopts a principle of continuous learning and training. Foremen, supervisors and site managers go through centralised training at the VSL Academy, where they learn best practice in all aspects of post-tensioning.

Barcelona New Exhibition Centre, Spain – 2008

VSL Post-Tensioning Systems

The VSL Post-Tensioning technology includes several systems that are specifically designed for different applications. The following table describes broadly these different systems and their main field of applications, which are thereon developed in this brochure.

	Monostrand and slab tendons		Multistrand tendons		Stressbar	
	Bonded	Unbonded	Internal	External	Internal	External
APPLICATIONS	• Slabs on grade • Building slabs • Transverse post-tensioning in bridge decks…		• Longitudinal post-tensioning in bridges • Building frames • Containments • Special structures…		• Short tendons, such as transverse post-tensioning for cable-stayed bridge pylons • Precast connections • Structural Strengthening…	

TIONS TOGETHER

R&D: THE KEY TO QUALITY AND DURABILITY

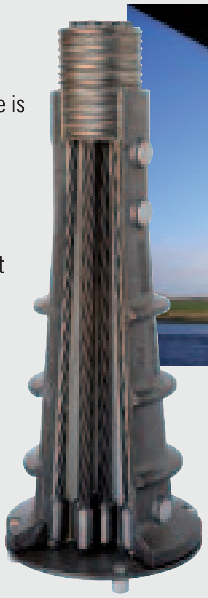
Research and development are VSL's driving force. The issues of QSE and sustainability have long been priorities together with the efficiency of construction methods and site works. This is also the case for post-tensioning products and services where durability, monitoring and inspection are important to focus on, as too are competence in design and methods.

Traceability and site efficiency

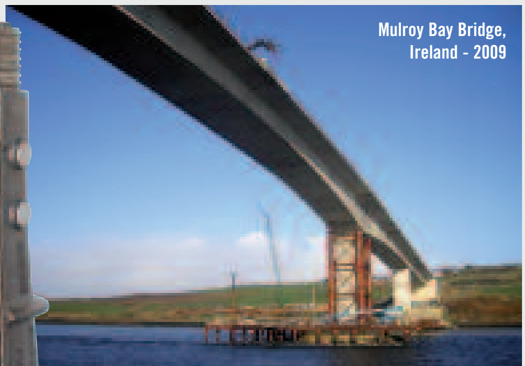
ADAPT, the tool for Automatic Data Acquisition for Post-Tensioning, collects data about tendon forces and elongation during stressing. It uses a personal digital assistant (PDA) to process the information for further use by the client.

PT Observer uses barcode process technology to collect all data throughout the entire post-tensioning process, assuring traceability. VSL's PT Observer and ADAPT systems greatly enhance the quality of the operational process.

Adaptable and cost-saving solutions



The VSL AF Anchorage is used for vertical tendons, where the prestressing force is transferred to the structure at its lowest end and where there is no access.



Mulroy Bay Bridge, Ireland - 2009

VSL develops custom-made specialised equipment such as movable scaffolding systems, launching girders, … for bridge construction and has the in-house capabilities to customise them from one project to the next.

New solutions for enhanced durability

Leak-tight encapsulation with PT-PLUS®

VSL continuously drives durability development and markets its PT-PLUS® plastic duct system for leak-tight encapsulation and higher fatigue resistance.

Electrical isolation with VSL CS 2000

Together with the CS 2000 Anchorage, PT-PLUS® ducts produce electrically-isolated tendons (EIT) and allow monitoring of the effectiveness of the corrosion-protective encapsulation. The same principle had already been a success with a VSL world-first, the use of electrically isolated ground anchors on a project in 1985.

Void control with the VSL Grout Void Sensor

The VSL Grout Void Sensor is installed at potentially critical points on a tendon and checks for the existence of voids after grouting.

Load control with the VSL Single Strand Load Cell

The VSL-designed Single Strand Load Cell allows economical and precise measurement of the load on a strand. It is compact and easy to install, fitting onto any VSL Anchor Head.

CREATING SOLUT

TRAINING: AT THE HEART OF STRONG PERFORMANCE

VSL is committed to investing in its staff, setting up training schemes and striving for professionalism.

VSL Academy

VSL has launched the VSL Academy to strengthen the company culture and to develop knowledge sharing by formalising and standardising the training of all post-tensioning foremen, supervisors and site engineers.

The goals of the VSL Academy are to:

- provide a unique training facility and tools within VSL to train our personnel in the skill and techniques required to perform the work to the highest standards specified today;
- provide hands-on practical training on post-tensioning mock-ups designed to cover all operational procedures;
- harmonise working procedures and enhance knowledge.

VSL Academy: a market leader's initiative

PMX – training in project management excellence

The programme's content combines technical topics, planning, organisation, risk management and result orientation with communication topics and leadership. Through this, VSL's managers transfer the fundamentals and culture of the company while promoting exchanges and useful networking throughout the group.

On site training

As a specialist contractor, VSL aims to maintain and develop its staff's skills on a long-term basis. Senior staff members are in charge of teaching VSL Techniques to new recruits.
A well-trained staff is VSL's most valuable asset in providing the best-possible service to clients.

VSL POST-TENSIONING SOLUTIONS FOR

Internal tendons – the most commonly-used solution

The VSL Systems are based on the method of post-tensioning. Most applications of the multi-strand system are internal and cement grouted, providing bond to the structure. Such tendons are extensively used in bridges and transportation structures as well as being applied successfully in building construction.

VSL's experience: 150,000 precast segments forming 6.3 million m² of bridge deck over the last 20 years

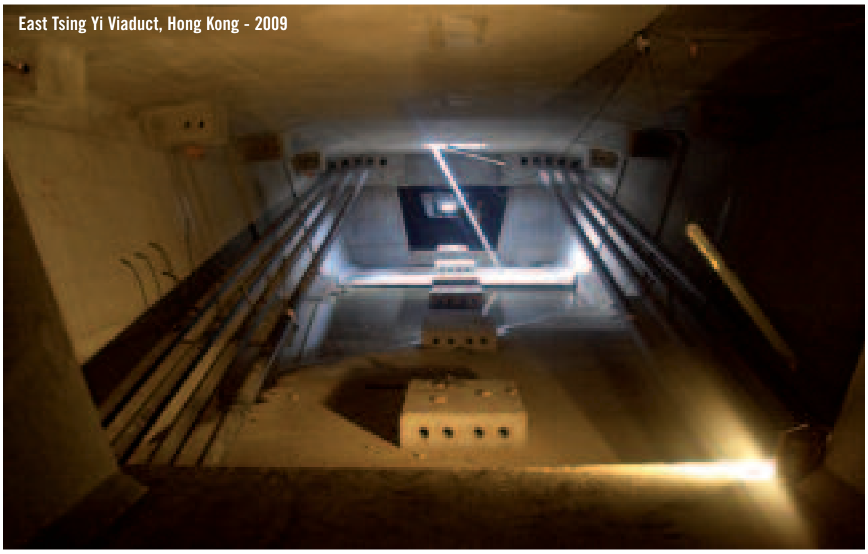
Shenzhen Western Corridor, Hong Kong - 2004

Kingston's Hunt's Bay Bridge, Jamaica - 2006

Gateway Upgrade, Australia - 2009

VSL Post-Tensioning systems lead and shape the state-of-the-art in bridge construction. They meet the advanced technical and practical requirements of today's engineers and construction professionals they are versatile and provide clients with unmatched durability, with a choice of steel or VSL PT-PLUS® plastic duct, as well as the availability of technical and site expertise for fully-encapsulated and electrically-isolated tendons (EIT). The systems comply with national and international standards and are approved by EOTA (European Organisation for Technical Approvals) and by other approval bodies.

BRIDGES


East Tsing Yi Viaduct, Hong Kong - 2009

External tendons for more flexibility

External post-tensioning tendons are positioned outside of the concrete section, though anchored into buttresses or diaphragms that form part of the bridge structure. They are therefore not bonded to the structure.
VSL External Post-Tensioning provides features such as the possibility of replacing tendons if required and easy inspection of the integrity of the corrosion protection. Applications are not restricted to concrete, but also include structural steel, composite steel-concrete bridges, timber and masonry structures. The external tendon technology has been used for bridge superstructures, girders in buildings and roof structures as well as for circular structures such as silos and reservoirs.


Boulonnais Viaduct, France - 1996


Gautrain Rapid Rail Link,
South Africa - 2009


Brunswick Head, Australia - 2006


Medway Crossing Bridge,
UK - 2001

External post-tensioning tendons can also be installed after completion of a structure if additional load capacity is required. This is done by adding tendons to the structure if the original design and construction were made to accommodate such an addition. Otherwise, a retrofit method can be implemented, although this requires a high level of engineering for structural analysis.

VSL POST-TENSIONING SOLUTIONS FOR BRIDGES

VSL Post-tensioning – a tool for pushing the limits

Bridge construction without post-tensioning is unthinkable. It is even a prerequisite for most of today's methods and allows the fast bridging of large spans with aesthetically-pleasing results. VSL's competence is outstanding in all known bridge construction methods. It is unrivalled in precast segmental construction, a method particularly suited to building large structures rapidly and economically even and especially into congested urban environments.

VSL as your "know-how partner"

VSL's post-tensioning know-how originates from thousands of projects and starts with a fundamental understanding of economically-optimised bridge concepts. With its design and methodology teams, VSL provides engineers and contractors with expertise in building cost-effective, durable and tailored structures.

INCREMENTAL LAUNCHING METHOD

Sagarra-Garrigues water channel, Spain - 2008

ERECTION BY LIFTING FRAME

West Tsing Yi Viaduct, Hong Kong - 2004

BALANCED CANTILEVER CAST-IN-SITU

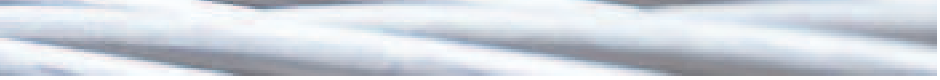
Gateway bridge upgrade, Australia - 2008

ERECTION BY UNDERSLUNG GANTRY

Windsor Floor Plane Project, Asutralia - 2006

ERECTION BY OVERHEAD GANTRY

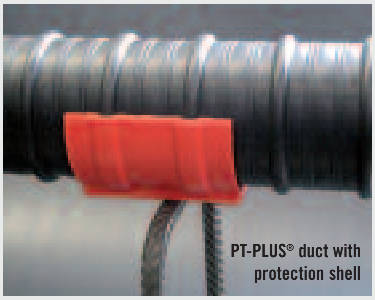
Metro de Santiago, Chile - 2005



ENHANCING DURABILITY

Gaining something extra with VSL's PT-PLUS® duct system

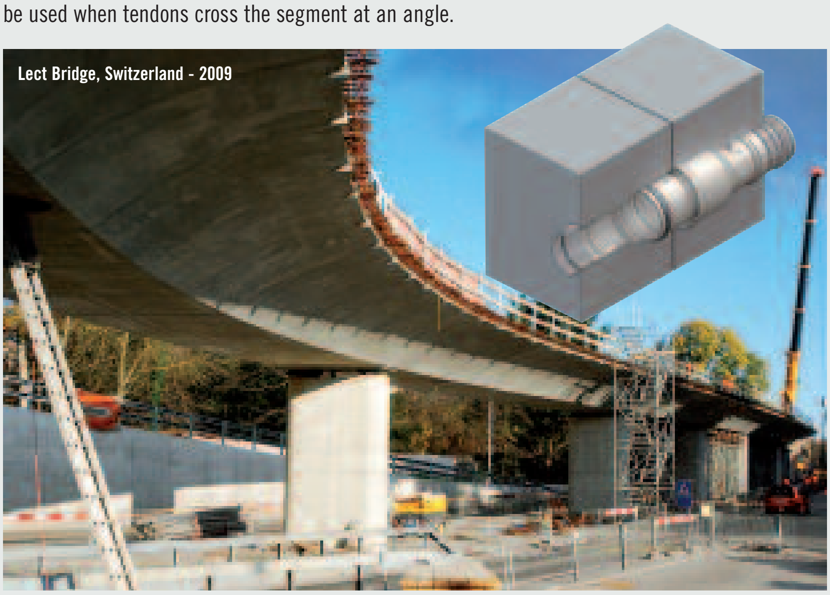
For conventional applications in non-agressive environment, corrugated steel ducts are normally used. However, the corrugated plastic ducts and plastic couplers of the VSL PT-PLUS® system provide important advantages when compared with conventional steel ducts, including tight encapsulation, high fatigue resistance and a low friction coefficient. For details see page 22.



PT-PLUS® duct with protection shell

A new coupler for EIT in precast structures

A new plastic coupler now permits full tendon encapsulation or EIT protection at the joints of precast segmental structures. The coupler is compact and similar in size to the ducting and can be used when tendons cross the segment at an angle.



Lect Bridge, Switzerland - 2009

Enhancing durability – VSL's concept for multi-layer protection

The multi-layer corrosion protection system enhances durability. It combines a careful overall concept and design of the structure's waterproof membranes, low-permeability concrete and leak-tight tendon encapsulation with a cementitious grout or other protection systems.

VSL is well qualified to assist decision makers with the adequate service when crucial protection strategies and measures are evaluated and decided.

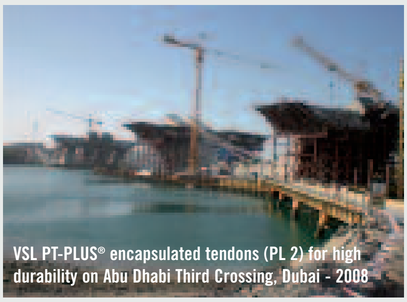
Electrically Isolated Tendons (EIT) at the Roeti Bridge, Switzerland - 2007

The tendon encapsulation - the decisive choice

Bearing in mind *fib*'s bulletin 33 and given the specific characteristics of PT-PLUS®, the following is recommended:

PL 1: using corrugated metal duct with special high quality grout (e.g. VSL's HPI Grouting). Cement grout provides excellent protection however grouting is a task for specialists. As an experienced specialist contractor, VSL carries out high-quality grouting using trained personnel and reliable equipment and in accordance with well proven procedures. In addition, VSL recommends the use of vacuum-assisted grouting for the most challenging conditions, such as where high points are not accessible or in other special cases. VSL provides a full service for this state-of-the-art technique.

PL 2: using PT-PLUS® ducts as leak tight encapsulation for enhanced protection against corrosion and fatigue, this is particularly suited for transverse tendons in bridge deck slabs and other structures where tendons are close to the concrete surface and subjected to fatigue; generally structures in severe corrosion environment and to bridges and other structures with fatigue loadings.



VSL PT-PLUS® encapsulated tendons (PL 2) for high durability on Abu Dhabi Third Crossing, Dubai - 2008

PL 3: allowing monitoring of the integrity of tendon encapsulation including protection against stray currents, applying the Electrical Isolation Tendon (EIT) method with PT-PLUS® ducts and the appropriate VSL Anchorage. VSL's grout void sensors enhance quality monitoring during grouting of tendons.

PL = Protection Level

VSL POST-TENSIONING IN BUILDINGS - A TOOL TO ACHIEVE SUBSTANTIAL BENE

Kens Project, Australia - 2004

Considerable savings for all parties

The advantages of using post-tensioning in buildings are being exploited in many countries and acknowledged by all partners in the construction process.

Owners benefit from:

- savings in materials in structures and foundations, leading to more economical construction;
- reduced financing costs due to shorter construction periods;
- less need for maintenance because of the crack and vibration control;
- more useable space within the available height limits;
- reduced deflection of structures.

Architects have:

- more aesthetic freedom and larger column-free spaces that generate more flexibility for offices, shopping centres, warehouses, car parks and similar structures.

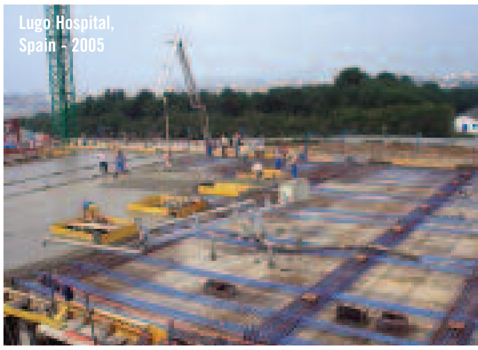
Contractors gain through:

- shorter construction time as formwork is often simpler and due to lesser back-propping;
- reduced cycle times as post-tensioning allows the structure to be stripped earlier leading to an overall reduction in the construction programme;
- fast and easy installation of electric, air conditioning and other services for flat slabs;
- less energy consumption.

WHotel, USA - 2006

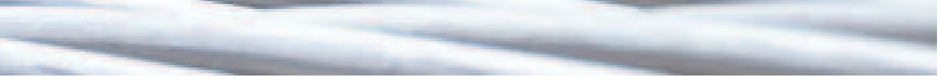
VSL's experience of economical applications:

- post-tensioned slabs for all types of buildings, parking structures and warehouses;
- post-tensioned transfer beams and transfer plates to provide spacious, column-free, architecturally pleasing spaces such as entrance halls, lobbies and convention rooms;
- post-tensioned raft foundations resulting in more economical solutions with improved deflection behaviour and better shear soil pressure distribution;
- post-tensioned concrete walls such as cores and masonry walls, allowing the architect and engineer to design with more flexibility and pleasing aesthetics;



Lugo Hospital, Spain - 2005

B2B Hotel, Mexico - 2008



FITS

- post-tensioning in structural members such as the mega-trusses of high-rise buildings to withstand wind-generated overturning moments.

VSL Post-tensioning Services – providing a solid frame for any structure

VSL's scope of services goes beyond the supply of components and includes:

- design assistance at the conceptual stage to select the best option for the floor system and provide preliminary sizing and quantities;

Burj Residence Dubai, UAE - 2007

- assistance in all detailed design stages with a constant aim of optimising savings in materials, achieving sustainability of the structure and easing construction to reduce the cycle times and the resources required;
- all works for the supply and installation of the post-tensioning materials, including a turnkey service package provided by VSL's site teams.

VSL's experience: Millions of square metres designed and built throughout the past 50 years

Detailed information is given in VSL's "Post-tensioning in building" publication (Report Series 4.1 and 4.2).

APPLICATIONS

FRAME CONSTRUCTION

Venetian Macao Resort Hotel, China - 2007
Frame construction for speed and ease of building with large open spaces or heavy loads.

SLAB CONSTRUCTION

RCBC Plaza, Philippines - 2000
VSL Post-Tensioning allows thinner slabs or larger spans.

MEGA-TRUSSES

International Commerce Centre, Hong Kong - 2008
VSL Post-Tensioning is often part of major structures such as this 480m-tall skyscraper, which is stabilised against typhoon winds through the use of post-tensioned mega-trusses that link the external columns to the inner core of the building.

TRANSFER PLATE/BEAMS CONSTRUCTION

Liverpool Tower, UK - 2006
Accommodating different floor layouts to ensure proper load transfer.

TIONS TOGETHER

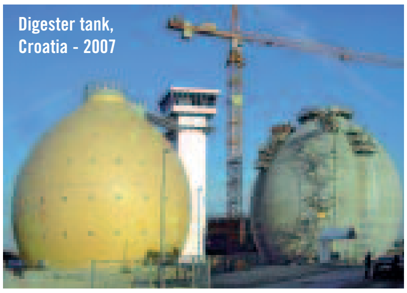
VSL POST-TENSIONING: IDEALLY SUITED

Unique VSL Anchorages for economical solutions

The shapes and functions of containment structures make them ideally suited to post-tensioning. Well-designed structures are practically crack-free and, most importantly, they are economical.

Water tank,
Cote d'Ivoire - 2008

Boyer Tank,
Australia - 2008



Digester tank,
Croatia - 2007

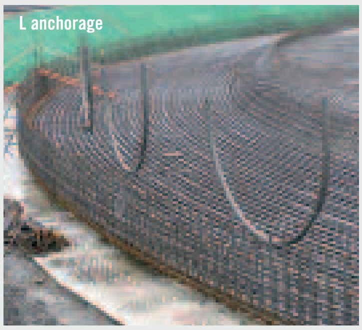
N'Kossa Barge,
France - 1995

ICM Corn Silo,
USA - 2008

Thanks to the variety of its post-tensioning anchorage systems, VSL offers versatile solutions for engineers and contractors to optimise costs and construction times. Some of the well-known VSL Anchorages are particularly suited for use in containment structures:

- The patented AF anchorage, which is used as the lower non-stressing anchorage for vertical tendons that are not accessible during strand installation and stressing;
- The L anchorage, which is used as the lower non-stressing anchorage for vertical tendons and allows the strand bundle to be pushed or pulled through and stressed after the concrete work for the wall has been finished;
- The Z anchorage, which is normally used for hoop tendons that can be installed within the wall thickness and which therefore do not necessitate buttresses for the stressing operation.

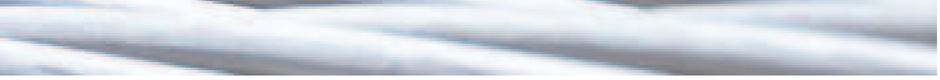
AF anchorages



L anchorage

Z anchorage

CREATING SOLU



TO CONTAINMENT STRUCTURES

Uljin Nuclear Power Plant, Korea - 2005

Meeting stringent requirements with exceptional reliability

Some applications are extraordinary and call for additional measures and special testing:

Nuclear applications

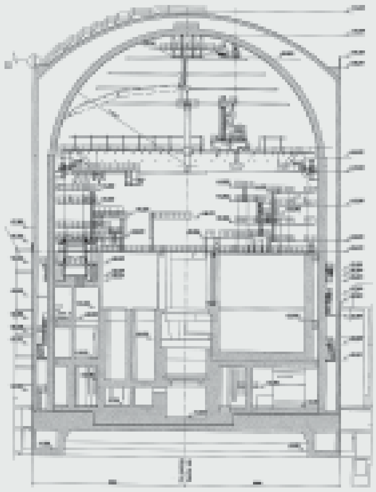
VSL carried out comprehensive tests on a full-scale mock-up of the latest generation of nuclear power plants to verify compliance of its PT systems and methods with new specific requirements. The purpose-built ring structure in Gien, France, has a radius of 24.46m and a height of 2.75m. VSL demonstrated that its systems, equipment and procedures meet the stringent requirements for installation, stressing and grouting operations on various types of tendons forming full 360° circle.

Liquefied gas applications

The construction of tanks for LNG and LPG (liquefied natural and petroleum gas) requires cryogenic testing of the post-tensioned tendons. During these tests, strands and anchorages are subjected to temperatures down to -196°C and are tested according to ETAG or other international standards. Through its long experience and proven post-tensioning systems, VSL is in a position to supply its post-tensioning systems to any LNG or LPG project around the globe.

Two units LAES-2 Nuclear Power Station in St Petersburg, Russia.

The VSL System with 55 greased and sheathed 0.6" strands is used for the 67.7m high inner of the two containment shells. 76 hoop tendons anchored in one of the two buttresses, 13 extra tendons in the dome, as well as 50 vertical over-the-dome tendons stressed from a stressing gallery are post-tensioned according to the latest nuclear containment requirements. The system allows checking the residual load, retensioning or replacing the tendons.



EXTREME TEMPERATURE TESTING

Anchorages and tendons tested at temperatures down to -196°C

Shanghai LNG Tank, China - 2007-2009

TIONS TOGETHER

VSL POST-TENSIONING FOR SLAB ON GRADE CONSTRUCTION: THE COST-EFFECTIVE SOLUTION


Wal-Mart, Mexico - 2007


Hangar Mosnov, Czech Republic - 2007

Benefits to the owner

Elimination of joints: Owners and operators benefit from the elimination of all or most of the costly joints, when using post-tensioned slab on grade.

Shorter construction time: Compared with ordinary reinforced concrete slabs, the use of VSL's technologies leads to less excavation, a thinner slab, little or no reinforcement and few if any joints. Large areas in excess of 2,500m^2 can be concreted, which results in a shorter construction time and contributes to a very competitive initial cost.

"Crack free" performance: Initial stressing can prevent shrinkage cracking. Post-tensioning compresses the slab and counteracts tensile stresses that would otherwise cause cracking under the worst combinations of loads or in poor soil conditions.

High impact and abrasion resistance: The compression resulting from post-tensioning combined with an optimum concrete strength and surface treatment reduces general wear and tear and subsequent maintenance costs.

Low maintenance: The significant reduction in the number of joints means that less maintenance is required, giving great improvements in operational efficiency.

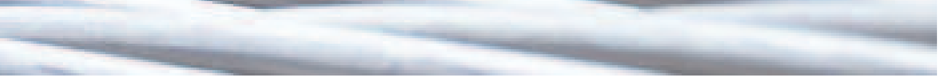

Austrak, Australia - 2003

Large slabs, indoor or outdoor

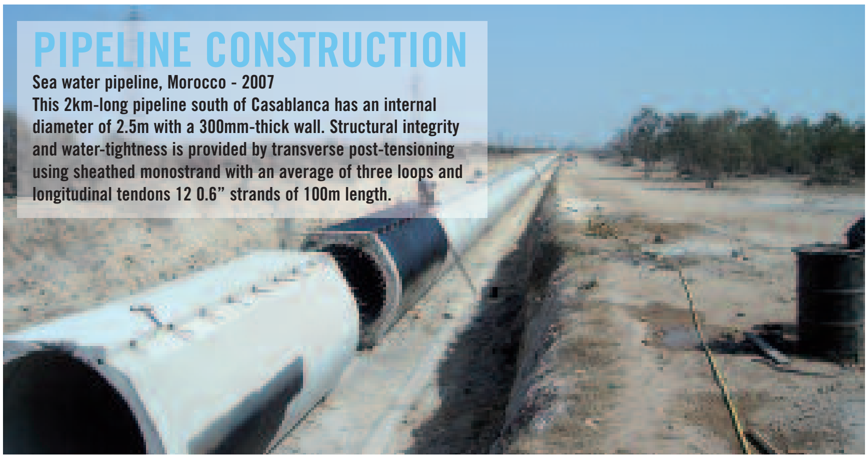
VSL Post-tensioning is widely used in the construction of pavement areas and in slabs on grade, where a concrete slab foundation is placed directly on the ground. Its advantages provide benefits in many different types of projects including warehouses, distribution centres, container storage terminals, rail and shipping terminals, airports, manufacturing facilities and as floor bases for liquid retaining structures. Post-tensioned slabs are also used for residential purposes and in recreation, such as for tennis courts and skating rinks.
VSL can provide the full range of services from the installation of post-tensioning to the complete design and construction of the concrete slab.


Nestlé Distribution Plant, Chile - 2001

The 30,000m^2 of joint-free slab of the Nestlé Plant constructed by VSL Chile represents the present world record.



VSL POST-TENSIONING FOR SPECIAL STRUCTURES: A SMART ALTERNATIVE



PIPELINE CONSTRUCTION

Sea water pipeline, Morocco - 2007
This 2km-long pipeline south of Casablanca has an internal diameter of 2.5m with a 300mm-thick wall. Structural integrity and water-tightness is provided by transverse post-tensioning using sheathed monostrand with an average of three loops and longitudinal tendons 12 0.6” strands of 100m length.

Versatile applications

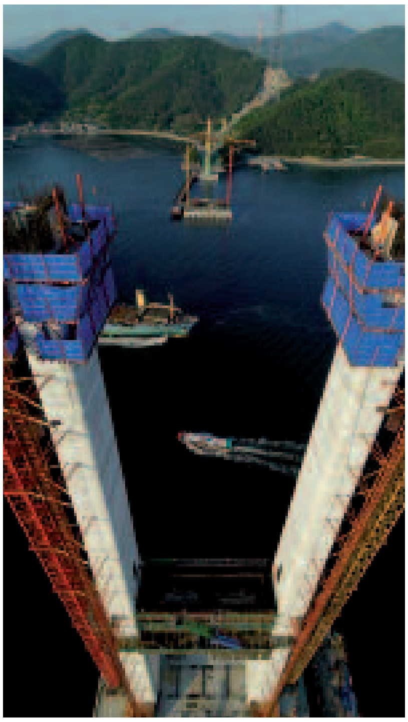
Without post-tensioning, many special structures could only be built with great effort, if indeed they could be built at all. Over the years, VSL's post-tensioning services have been used for a very wide range of highly prestigious and complex structures including offshore platforms, concrete floating barges, dams and many others. Customers value the experience and versatility they gain by having VSL as a partner from the early planning stages through to construction.

STADIUM CONSTRUCTION

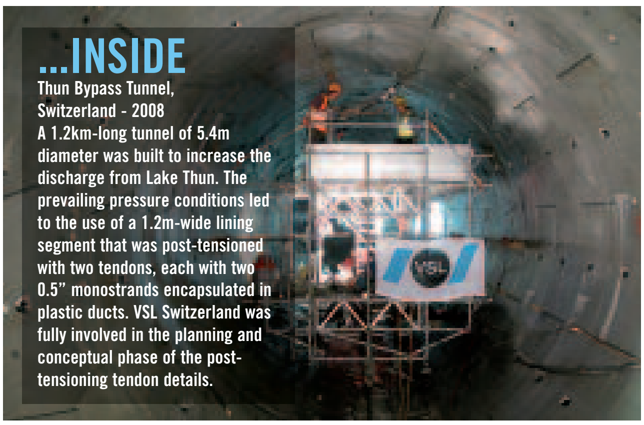
José Alvalade Stadium, Portugal - 2004
The challenge of building a multifunctional stadium with two halls that are part of irregular and complicated structural elements is an excellent example of a project where clients can benefit from the versatility of VSL as a professional post-tensioning partner.

SUB-STRUCTURE CONSTRUCTION

Machang Bridge, Korea - 2006
The cable-stayed Machang Bridge crosses the neck of the Masan Bay. Post-tensioning tendons with VSL Loop Anchorages were installed for the deck-to-pile footing tie-down system in the piers supporting the bridge's back spans.

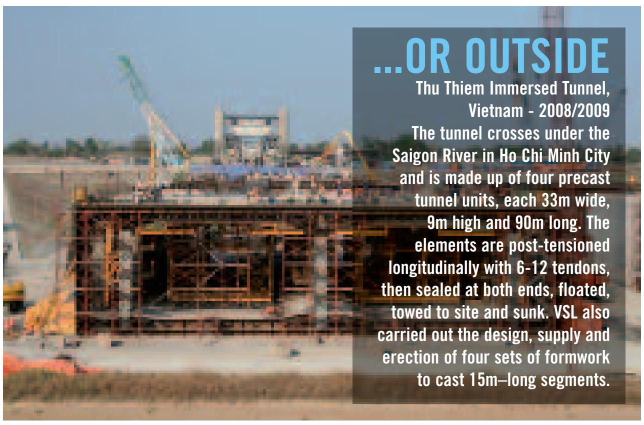


TUNNEL CONSTRUCTION - WHETHER HYDROSTATIC PRESSURE PUSHES FROM...



...INSIDE

Thun Bypass Tunnel, Switzerland - 2008
A 1.2km-long tunnel of 5.4m diameter was built to increase the discharge from Lake Thun. The prevailing pressure conditions led to the use of a 1.2m-wide lining segment that was post-tensioned with two tendons, each with two 0.5” monostrands encapsulated in plastic ducts. VSL Switzerland was fully involved in the planning and conceptual phase of the post-tensioning tendon details.



...OR OUTSIDE

Thu Thiem Immersed Tunnel, Vietnam - 2008/2009
The tunnel crosses under the Saigon River in Ho Chi Minh City and is made up of four precast tunnel units, each 33m wide, 9m high and 90m long. The elements are post-tensioned longitudinally with 6-12 tendons, then sealed at both ends, floated, towed to site and sunk. VSL also carried out the design, supply and erection of four sets of formwork to cast 15m–long segments.

VSL POST-TENSIONING FOR REPAIR WO
A MUST FOR TAILOR-MADE SOLUTIONS

Structural remedial work requires thorough diagnosis of damage and deterioration followed by full assessment of the causes, risks and consequences involved.
VSL employs state-of-the-art equipment and special inspection techniques to detect defects in reinforced and prestressed concrete structures before any significant damage occurs. Close co-operation with materials testing institutes and structural designers, together with the use of the latest investigation techniques, enables VSL to prepare precise and comprehensive reports.

Assessment diagnosis of structural conditions includes:

- inspection and surveillance of concrete structures;
- condition evaluation of the same;
- root cause analysis;
- design of repair strategies;
- estimating the order of magnitude for the cost of repairs.

REPLACEMENT OF EXTERNAL POST-TENSIONING IN BRIDGES

St Cloud Viaduct, France - 2000

The external tendons that reinforced the 1974-built 1,102m-long Saint Cloud Bridge between Paris and Normandy showed signs of corrosion and the client decided to replace them. As a first precautionary step, shock-absorbers were fitted at each side of the deviators before the tendons were cut and the anchorages removed or adapted. New external tendons were then installed by VSL.

REPAIR OF BRIDGES

Figueira de Foz Bridge, Portugal - 2005

VSL Portugal, in partnership with a local contractor, carried out repair works including external post-tensioning, strengthening of the abutments with bars and replacement of expansion joints. There was also retrofitting of structural bearings and seismic devices, including the installation of 4 x 500kN shock-absorbers at the abutments.

RKS:

STRENGTHENING OF HISTORICAL BUILDINGS

“Las Arenas” Bullfighting Ring, Spain - 2007

One of the many examples in Barcelona where VSL has assisted with engineering and specialised site works is this former bull ring, built in 1898, which has been transformed into a leisure and entertainment complex. VSL carried out engineering and post-tensioning works in connection with the transfer slab and beams of the Neo-Mudéjar façade. The project involved post-tensioned floors with spans of between 12m and 17m and the supply of other VSL products such a neoprene bearings and studs.

The Leaning Tower of Pisa, Italy - 1993

VSL strengthened the world-renowned Leaning Tower of Pisa with 18 specially-developed monostrand hoop tendons. The optimum solution consisted of a marble-coloured PE-sheath and galvanized, non-greased 0.6" strand with a centre stressing anchorage, allowing force adjustment and monitoring during and after the stressing operation.

STRENGTHENING OF A NUCLEAR POWER PLANT

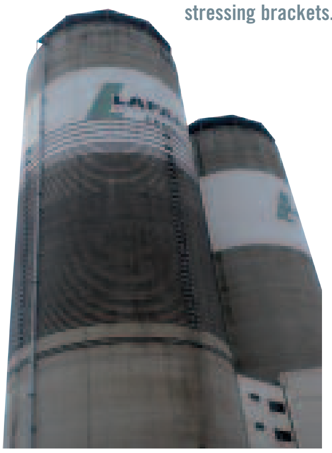
Gösgen Nuclear Power Plant, Switzerland - 2005

A carbon fibre tendon system was used for the seismic upgrade of the emergency feed building at the Gösgen nuclear power plant. The system consists of carbon CFRP plates and head and is well suited for seismic and other strengthening measures where post-tensioning forces are needed in very thin tensile members.

SILO REPAIR AND STRENGTHENING

Blue Circle Cement Silo, Singapore - 2001

The 60m-tall silo was strengthened using a VSL-engineered solution of externally wrapped, bonded tendons each with four strands of 0.6”. The 66 tendons are encapsulated in flat high-density polyethylene ducts and anchored into special stressing brackets.



VSL’s other repair solutions

VSL also provides other structural solutions for the repair and strengthening of structures including:

- passive strengthening with the design and application of:
  - bonded CFRP (carbon fibre reinforced polymer);
  - bonded SRP (steel reinforced polymer).
- protection with:
  - Ductal®, the ultra-high-strength and ductile blast-resistant solution;
  - dampers for mitigation of vibration induced by earthquake, wind and human activities;
  - cathodic protection for corrosion mitigation.

TIONS TOGETHER

CREATING SOLUTIONS TOGETHER

GROUND ENGINEERING

- Ground anchors
- VSoL® walls
- D-walls & Piles
- Ground improvement

CONSTRUCTION

- Bridges
- Buildings
- Slab on grade
- Containment structures
- Special structures
- Heavy lifting
- Formwork & Equipment

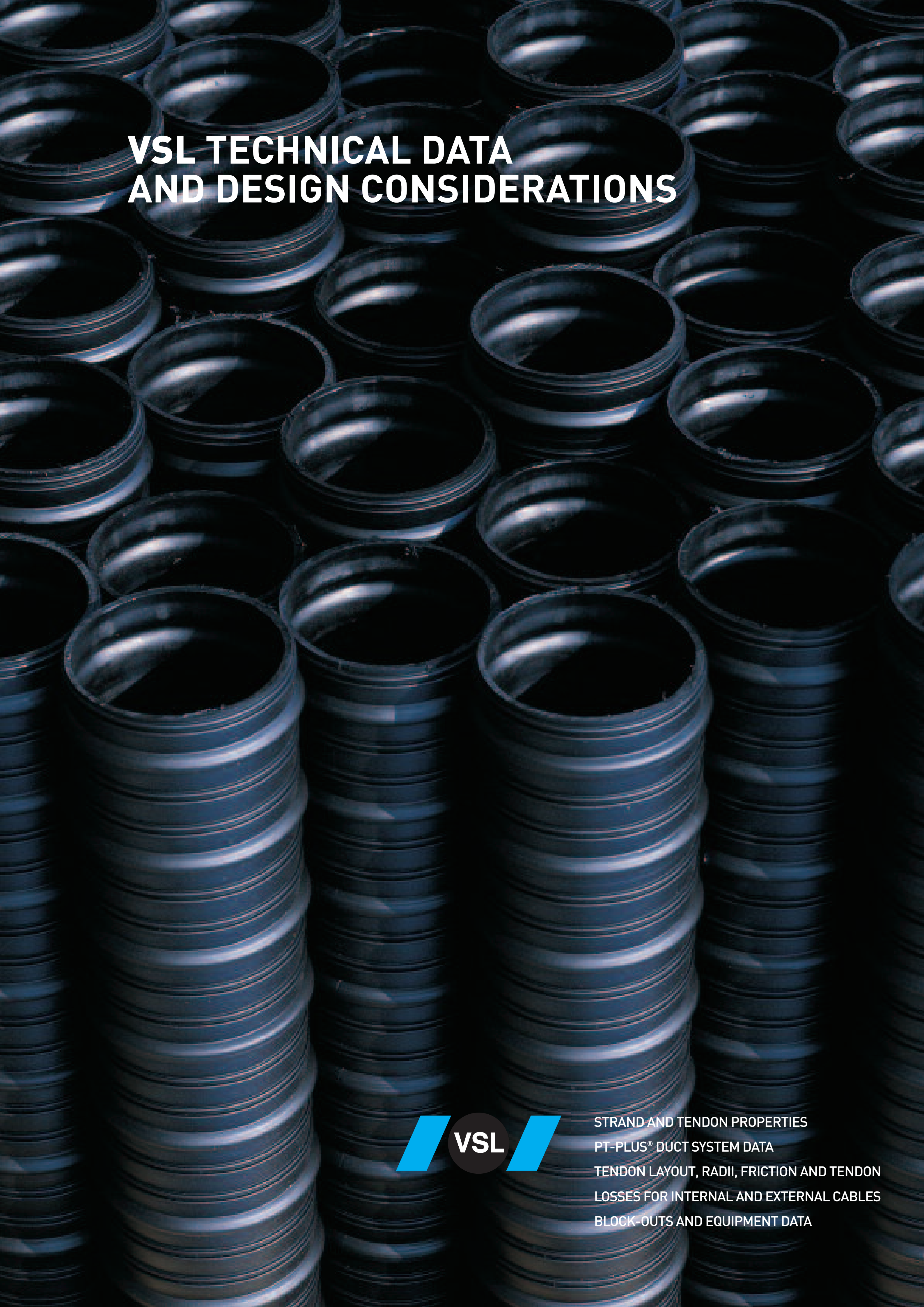
REPAIR, STRENGTHENING & PRESERVATION

- Structural diagnostics & Monitoring
- Repair & Strengthening
- Protection & Preservation

SYSTEMS & TECHNOLOGIES

- Post-tensioning strand systems
- Bars & post-tensioning bar systems
- Stay cable systems
- Damping systems (stays & buildings)
- Ductal® UHP concrete
- Bearings & Joints

www.vsl.com



VSL TECHNICAL DATA AND DESIGN CONSIDERATIONS

VSL

STRAND AND TENDON PROPERTIES

PT-PLUS® DUCT SYSTEM DATA

TENDON LAYOUT, RADII, FRICTION AND TENDON LOSSES FOR INTERNAL AND EXTERNAL CABLES

BLOCK-OUTS AND EQUIPMENT DATA

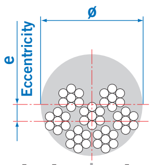
1 - STRAND

1.1 - STRAND PROPERTIES 13mm (0.5")

Strand type			prEN 10138 – 3 (2006) Y1860S7		ASTM A 416-06 Grade 270
Nominal diameter	d	(mm)	12.5	12.9	12.7
Nominal cross section	Ap	(mm²)	93	100	98.7
Nominal mass	M	(kg/m)	0.726	0.781	0.775
Nominal yield strength	$f_{p0,1k}$	(MPa)	1634[1]	1640[1]	1675[2]
Nominal tensile strength	f_{pk}	(MPa)	1860	1860	1860
Specif./min. breaking load	F_{pk}	(kN)	173	186	183.7
Young's modulus		(GPa)	approx. 195		
Relaxation[3] after 1000 h at 20°C and 0.7 x F_{pk}		(%)	max. 2.5		

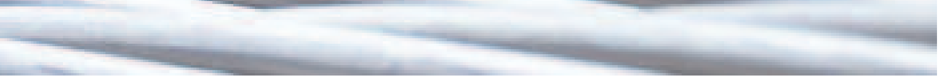
1) Characteristic value measured at 0.1% permanent extension
2) Minimum load at 1% extension for low-relaxation strand
3) Valid for relaxation class acc. to prEN 10138-3 or low-relaxation grade acc. to ASTM A 416-06

1.2 - TENDON PROPERTIES 13mm (0.5")



Unit	Strands numbers	Steel area			Breaking load			Corrugated steel duct[3] (recommended)		Corrugated plastic duct VSL PT-PLUS®		Steel pipes
		Ap acc. to prEN		ASTM	Y1860S7 (prEN)		Grade 270 (ASTM)	Øi / Øe	e	Øi / Øe	e	Ø ext x t
		d=12.5 mm Ap=93 mm²	d=12.9 mm Ap=100 mm²	d=12.7 mm Ap=100 mm²	d=12.5 mm Ap=93 mm²	d=12.9 mm Ap=100 mm²	d=12.7 mm Ap=98.7 mm²					
		[mm²]	[mm²]	[mm²]	[kN]	[kN]	[kN]	[mm]	[mm]	[mm]	[mm]	[mm]
5-1	1	93	100	98.7	173	186	183.7	20/25	3	22/25	6	25.0 x 2.0
5-2	2	186	200	197	346	372	367	35/40	8	76/25[2]	-	31.8 x 2.0/2.5/3.0
5-3	3	279	300	296	519	558	551	35/40	6	76/25[2]	-	33.7 x 2.0/2.5/3.0
5-4	4	372	400	395	692	744	735	40/45[1]	7	76/25[2]	-	42.4 x 2.0/2.5/3.0
5-7	5	465	500	494	865	930	919	45/50	8	58/63	14	60.3 x 2.0/2.5/3.0
	6	558	600	592	1038	1116	1102	45/50	6	58/63	12	
5-7	7	651	700	691	1211	1302	1286	50/57	7	58/63	11	60.3 x 2.0/2.5/3.0
5-12	8	744	800	790	1384	1488	1470	55/62	9	58/63	10	70.0 x 2.0/2.5/3.0
	9	837	900	888	1557	1674	1653	55/62	8	58/63	9	
	10	930	1000	987	1730	1860	1837	60/67	10	58/63	9	
	11	1023	1100	1086	1903	2046	2021	60/67	9	58/63	8	
5-12	12	1116	1200	1184	2076	2232	2204	60/67	8	58/63	7	70.0 x 2.0/2.5/3.0
5-15	13	1209	1300	1283	2249	2418	2388	65/72	9	76/81	14	82.5 x 2.0/2.5/3.0
	14	1302	1400	1382	2422	2604	2572	65/72	8	76/81	13	
5-15	15	1395	1500	1481	2595	2790	2756	70/77	9	76/81	12	82.5 x 2.0/2.5/3.0
5-19	16	1488	1600	1579	2768	2976	2939	70/77	9	76/81	12	88.9 x 2.5/3.0/3.5
	17	1581	1700	1678	2941	3162	3123	75/82	11	76/81	11	
	18	1674	1800	1777	3114	3348	3307	75/82	10	76/81	10	
5-19	19	1767	1900	1875	3287	3534	3490	75/82	9	76/81	9	88.9 x 2.5/3.0/3.5
5-22	20	1860	2000	1974	3460	3720	3674	80/87	10	100/106	20	88.9 x 2.5/3.0/3.5
	21	1953	2100	2073	3633	3906	3858	80/87	9	100/106	19	
5-22	22	2046	2200	2171	3806	4092	4041	80/87	8	100/106	18	88.9 x 2.5/3.0/3.5
5-27	23	2139	2300	2270	3979	4278	4225	85/92	12	100/106	19	101.6 x 3.0/4.0/5.0
	24	2232	2400	2369	4152	4464	4409	85/92	11	100/106	18	
	25	2325	2500	2468	4325	4650	4593	90/97	14	100/106	19	
	26	2418	2600	2566	4498	4836	4776	90/97	13	100/106	18	
5-27	27	2511	2700	2665	4671	5022	4960	95/102	15	100/106	17	101.6 x 3.0/4.0/5.0
5-31	28	2604	2800	2764	4844	5208	5144	95/102	14	100/106	16	108.0 x 3.0/4.0/5.0
	29	2697	2900	2862	5017	5394	5327	95/102	13	100/106	15	
	30	2790	3000	2961	5190	5580	5511	95/102	12	100/106	14	
5-31	31	2883	3100	3060	5363	5766	5695	95/102	11	100/106	13	108.0 x 3.0/4.0/5.0
5-37	32	2976	3200	3158	5536	5952	5878	100/107	13	115/121	20	114.3 x 3.0/4.0/5.0
	33	3069	3300	3257	5709	6138	6062	100/107	12	115/121	19	
	34	3162	3400	3356	5882	6324	6246	100/107	12	115/121	19	
	35	3255	3500	3455	6055	6510	6430	110/117	17	115/121	19	
	36	3348	3600	3553	6228	6696	6613	110/117	17	115/121	19	
5-37	37	3441	3700	3652	6401	6882	6797	110/117	16	115/121	18	114.3 x 3.0/4.0/5.0
5-43	43	3999	4300	4244	7439	7998	7899	120/127	18	130/136	23	127.0 x 3.0/4.0/5.0
5-55	55	5115	5500	5429	9515	10230	10104	130/137	17	130/136	17	139.7 x 3.0/4.0/5.0

1) Flat ducts possible as well
2) Flat duct PT-PLUS with rectangular slab anchorages, for PT-PLUS see also under 3.1.3.
3) If flat ducts (steel or PT PLUS) to be used with square type castings please contact your VSL representative. In plan view, tendons with slab type anchorages must be straight between anchorages or have only unidirectional turns with min. radii of > 6 m. Strands must always be pushed-in prior to concreting. Eccentricity e: negligible
4) Given values may slightly vary depending on local availability of ducts. They are minimal for most applications. For special cases (long tendons, many curvatures, small radii etc.) greater size duct is recommended – please verify with VSL. In any case the filling ratio (cross-section steel / duct) must not exceed 0.5 (EN523).
5) Please check with the nearest VSL office for the complete anchorage list

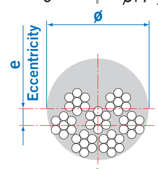


1.3 - STRAND PROPERTIES 15mm (0.6")

Strand type			prEN 10138 – 3 (2006) Y1860S7		ASTM A 416-06 Grade 270
Nominal diameter	d	(mm)	15.3	15.7	15.24
Nominal cross section	A_p	(mm²)	140	150	140
Nominal mass	M	(kg/m)	1.093	1.172	1.102
Nominal yield strength	$f_{p0,1k}$	(MPa)	1636[1]	1640[1]	1676[2]
Nominal tensile strength	f_{pk}	(MPa)	1860	1860	1860
Specif./min. breaking load	F_{pk}	(kN)	260	279	260.7
Young's modulus		(GPa)	approx. 195		
Relaxation[3] after 1000 h at 20°C and 0.7 x F_{pk}		(%)	max. 2.5		

1) Characteristic value measured at 0.1% permanent extension
2) Minimum load at 1% extension for low-relaxation strand
3) Valid for relaxation class acc. to prEN 10138-3 or low-relaxation grade acc. to ASTM A 416-06

1.4 - TENDON PROPERTIES 15mm (0.6")

Unit	Strands numbers	Steel area			Breaking load			Corrugated steel duct[3] (recommended)		Corrugated plastic duct VSL PT-PLUS®		Steel pipes
		A_p acc. to prEN		ASTM	Y1860S7 (prEN)		Grade 270 (ASTM)	Øi / Øe	e 	Øi / Øe	e	Ø ext. x t
		d=15.3 mm A_p=140 mm²	d=15.7 mm A_p=150 mm²	d=15.24 mm A_p=140 mm²	d=15.3 mm A_p=140 mm²	d=15.7 mm A_p=150 mm²	d=15.24 mm A_p=140 mm²					
		[mm²]	[mm²]	[mm²]	[kN]	[kN]	[kN]	[mm]	[mm]	[mm]	[mm]	[mm]
6-1	1	140	150	140	260	279	260.7	25/30	5	23/25	4	25.0 x 2.0
6-2	2	280	300	280	520	558	521	40/45	9	76/25[2]	-	33.7 x 2.0/2.5/3.0
6-3	3	420	450	420	780	837	782	40/45	6	76/25[2]	-	42.4 x 2.0/2.5/3.0
6-4	4	560	600	560	1040	1116	1043	45/50[1]	7	76/25[2]	-	48.3 x 2.0/2.5/3.0
6-7	5	700	750	700	1300	1395	1304	50/57	8	58/63	13	76.1 x 2.0/2.5/3.0
│	6	840	900	840	1560	1674	1564	55/62	9	58/63	11	│
6-7	7	980	1050	980	1820	1953	1825	55/62	7	58/63	9	76.1 x 2.0/2.5/3.0
6-12	8	1120	1200	1120	2080	2232	2086	65/72	11	76/81	18	80.0 x 2.0/.2.5
│	9	1260	1350	1260	2340	2511	2346	65/72	9	76/81	16	│
│	10	1400	1500	1400	2600	2790	2607	70/77	11	76/81	15	│
│	11	1540	1650	1540	2860	3069	2868	70/77	9	76/81	13	│
6-12	12	1680	1800	1680	3120	3348	3128	75/82	11	76/81	12	80.0 x 2.0/.2.5
6-15	13	1820	1950	1820	3380	3627	3389	80/87	13	100/106	25	101.6 x 3.0/4.0/5.0
│	14	1960	2100	1960	3640	3906	3650	80/87	11	100/106	24	
6-15	15	2100	2250	2100	3900	4185	3911	80/87	10	100/106	23	101.6 x 3.0/4.0/5.0
6-19	16	2240	2400	2240	4160	4464	4171	85/92	12	100/106	22	101.6 x 3.0/4.0/5.0
│	17	2380	2550	2380	4420	4743	4432	85/92	11	100/106	20	│
│	18	2520	2700	2520	4680	5022	4693	90/97	13	100/106	19	│
6-19	19	2660	2850	2660	4940	5301	4953	90/97	12	100/106	18	101.6 x 3.0/4.0/5.0
6-22	20	2800	3000	2800	5200	5580	5214	100/107	17	100/106	17	114.3 x 3.0/4.0/5.0
│	21	2940	3150	2940	5460	5859	5475	100/107	16	100/106	16	│
6-22	22	3080	3300	3080	5720	6138	5735	100/107	15	100/106	15	114.3 x 3.0/4.0/5.0
6-27	23	3220	3450	3220	5980	6417	5996	100/107	14	115/121	22	114.3 x 3.0/4.0/5.0
│	24	3360	3600	3360	6240	6696	6257	100/107	13	115/121	22	│
│	25	3500	3750	3500	6500	6975	6518	110/117	18	115/121	21	│
│	26	3640	3900	3640	6760	7254	6778	110/117	17	115/121	21	│
6-27	27	3780	4050	3780	7020	7533	7039	110/117	16	115/121	20	114.3 x 3.0/4.0/5.0
6-31	28	3920	4200	3920	7280	7812	7300	110/117	15	130/136	27	127.0 x 3.0/4.0/5.0
│	29	4060	4350	4060	7540	8091	7560	120/127	21	130/136	27	│
│	30	4200	4500	4200	7800	8370	7821	120/127	20	130/136	26	│
6-31	31	4340	4650	4340	8060	8649	8082	120/127	19	130/136	25	127.0 x 3.0/4.0/5.0
6-37	32	4480	4800	4480	8320	8928	8342	120/127	18	130/136	24	139.7 x 3.0/4.0
│	33	4620	4950	4620	8580	9207	8603	120/127	17	130/136	23	│
│	34	4760	5100	4760	8840	9486	8864	120/127	16	130/136	22	│
│	35	4900	5250	4900	9100	9765	9125	130/137	22	130/136	22	│
│	36	5040	5400	5040	9360	10044	9385	130/137	21	130/136	21	│
6-37	37	5180	5550	5180	9620	10323	9646	130137	20	130/136	20	139.7 x 3.0/4.0
6-43	43	6020	6450	6020	11180	11997	11210	140/147	21	150/157	27	152.4 x 3.0/4.0/5.0
6-55	55	7700	8250	7700	14300	15345	14339	160/167	26	150/157	21	168.3 x 3.0/4.0

1) Flat ducts possible as well
2) Flat duct PT-PLUS with rectangular slab anchorages, for PT-PLUS see also under 3.1.3.
3) If flat ducts (steel or PT PLUS) to be used with square type castings please contact your VSL representative. In plan view, tendons with slab type anchorages must be straight between anchorages or have only unidirectional turns with min. radii of > 6 m. Strands must always be pushed-in prior to concreting. Eccentricity e: negligible
4) Given values may slightly vary depending on local availability of ducts. They are minimal for most applications. For special cases (long tendons, many curvatures, small radii etc.) greater size duct is recommended – please verify with VSL. In any case the filling ratio (cross-section steel / duct) must not exceed 0.5 (EN523).
5) Please check with the nearest VSL office for the complete anchorage list

2 - ANCHORAGES

For the selection and the dimensions of the most commonly used anchorages, please consult the VSL data sheets on anchorages. For spacing between anchorages and edge distance, refer to individual anchorage data sheet.

3 - DUCTING

3.1 TYPES

3.1.1 Bright corrugated steel ducts

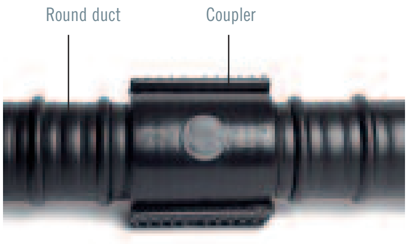
The most commonly used sheaths are made from rolled steel strip. Round and flat (max. 5 strands are available. They are corrugated and leak-tight and must have sufficient strength to withstand varying degrees and types of mechanical loading. For additional information and details, locally valid norms (or for example EN523) can be consulted.

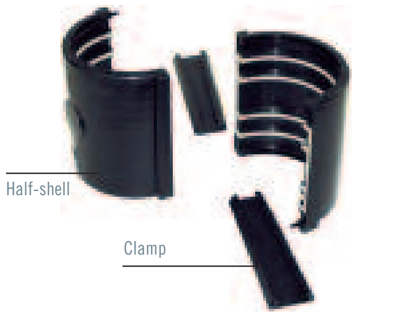
3.1.2 Galvanized corrugated steel ducts

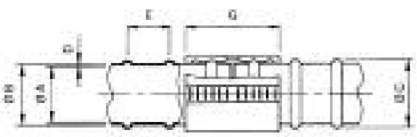
Galvanization is sometimes used to ensure corrosion protection of the metal strip. It can provide lower friction losses when stressing the tendon.
Please check local code requirements.

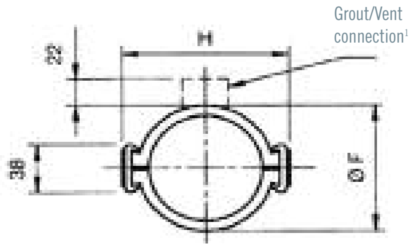
3.1.3 Corrugated PT-PLUS® duct system

For enhanced corrosion protection and fatigue resistance of the tendons, use of the VSL PT-PLUS® corrugated plastic duct system is recommended. The PT-PLUS® system is particularly suitable for railroad brides, bridge decks, parking structures and other situations where severe corrosion or high fatigue loading may be expected. In addition, the PT-PLUS® system with additional details at the anchorages allows to provide electrically isolated tendons (EIT) and a protection level of PL3 (*fib* bulletin 33). These EIT tendons permit monitoring of the leak tightness of the tendon encapsulation and protection of the tendon over the entire design life of the structure.









[1] *Couplers are available with grout / vent connections for a threaded tube with dia. 23 mm*

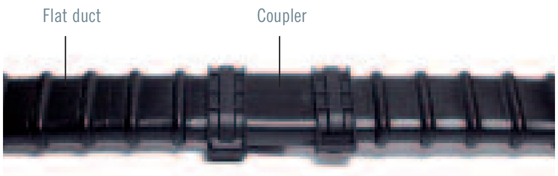
Dimensions for PT-PLUS® duct system

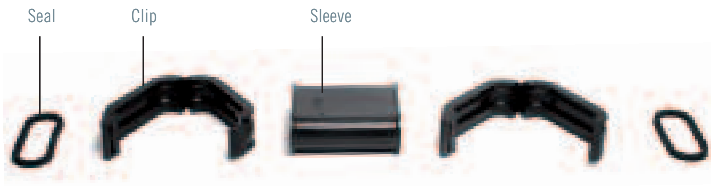
Dimensions in mm, subject to modifications

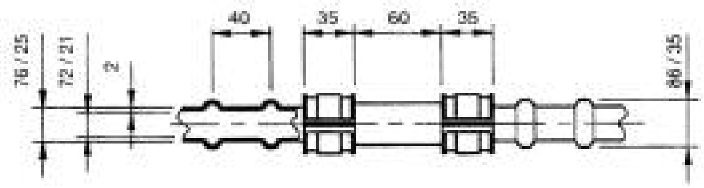
Type	Tendon unit		Ø A	Ø B	Ø C	D	E	Ø F	G	H	Nominal weight	
	0.5"	0.6"									Duct kg/m	Coupler kg/unit
22	5-1	6-1	22	25	31	1.5	55	27.5[2]	80[2]	27.5[2]	0.180	0.010
59	5-12	6-7	58	63	73	2.5	42	82	108	106	0.480	0.200
76	5-19	6-12	76	81	91	2.5	52.5	100	116	124	0.620	0.250
100	5-31	6-19/22	100	106	116	3.0	60	123	126	147	0.980	0.270
115	5-37	6-27	115	121	131	3.0	60	138	127	162	1.120	0.320
130	5-43/55	6-31/37	130	136	146	3.0	52	153	134	177	1.200	0.380
150		6-55	150	157	167	3.5	60	175	126	198	1.620	0.420

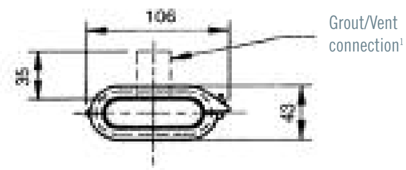
[2] *One-piece sleeve coupler*

Note: PT-PLUS ducts come in lengths of approximately 6 m, type 22 ducts are 7 m long









Dimensions in mm, subject to modifications

[1] *Couplers are also available with grout / vent connections for a smooth tube with dia. 16 / 20 mm.*

The PT-PLUS® flat duct system and type 22 are often used for slab post-tensioning in buildings, for transversal tendons for bridges and for similar structures where the exploitation of a maximum tendon eccentricity in relatively thin members is important.

3.1.4 Smooth plastic ducts

Smooth plastic ducts are predominantly used for external tendons. Occasionally they have been also used for internal tendons when no bonding steel / concrete is required. They are normally made of UV resistant, new high density polyethylene (HDPE) material (virgin granulate) acc. to EN12201 and ASTM D3035 or ASTM F714 or equivalent standards. Material recycled from previously used PE components shall not be used. Ducts normally have a ratio of diameter / wall thickness of 16 to 18, with an internal diameter not smaller than $1.7\sqrt{Ap}$ (Ap = nominal cross section of the steel area in the tendon), suitable to carry internal pressure during grouting (ETAG013 (2002) e.g. specifies 1 MPa / 10 bar design pressure). The following dimensions of external tendon pipes are recommended (see table below).

3.1.5 Steel pipes

In certain applications (e.g. cryogenic, nuclear, offshore) where the ducts are subject to high loading when particularly tight tendon curvature is required, or when tendons are in congested parts of structures, steel pipes are used. Tubes are thin (in compliance with EN or equivalent standards) and machine-bendable, recommended dimensions see 1.2 / 1.4.

Steel tubes used externally: dimensions are primarily dictated by the availability of local standardized tubes. The table below can serve as a guideline and is based on an internal diameter of $\geq 1.7\sqrt{Ap}$ where Ap represents the cross section of the prestressing steel, irrespective of whether the strand is bare or sheathed and greased.

Dimensions for smooth plastic ducts

Tendon size	External pipe diameter (mm)		Wall thickness (mm)	
	strands		strands	
	bare	PE sheathed	bare	PE sheathed
5-12 / 6-7	75	90	4.3	5.1
5-15/19 / 6-12	90	110	5.4	6.0
5-22/31 / 6-15/19	110	140	6.6	6.7
5-37 / 6-22/27	125	160	7.7	7.7
5-43 / 6-31	140	160	8.3	7.7
5-55 / 6-37	160	180	9.5	8.6

Dimensions in mm, subject to modifications

Dimensions for steel pipes

Strand Nos.	Min inside dia. for strands with		
	100 mm²	140 mm²	150 mm²
4	34	40.2	41.6
7	45	53.2	55.1
12	58.9	69.7	72.1
15	65.8	77.9	80.6
19	74.1	87.7	90.8
22	79.7	94.3	97.7
27	88.3	104.5	108.2
31	94.7	112.0	115.9
37	103.4	122.4	126.6
43	111.5	131.9	136.5
55	126.1	149.2	154.4

Dimensions in mm, subject to modifications

Wall thickness e ≥ Ø / 50 or minimum 1.5 mm
Ø = external diameter
Where steel pipes need to be welded, e ≥ 3 mm

3.2 FRICTION COEFFICIENT AND LOSSES DUE TO PRESTRESSING

3.2.1 Friction coefficient

The following values may be assumed when using the equation $P_x = P_0 \, e^{-(\mu\varphi_x + kx)}$:

Equation of loss of post-tensioning force along a tendon

P_x = Remaining force at distance x from the stressing end
P_0 = Stressing force at the stressing end
μ = Friction coefficient
$\mu\varphi_x$ = Accumulated tendon deviation from the stressing end
k = Wobble coefficient
x = Distance from the stressing end

	Range	Recommended value
Corrugated Steel Sheath	$\mu = 0.16 - 0.24$ $k = (0.6 - 1.0) \times 10^{-3}\ m^{-1}$	$\mu = 0.20$ $k = 0.8 \times 10^{-3}\ m^{-1}$
PT-PLUS® Plastic Duct	$\mu = 0.12 - 0.14$ $k = (0.8 - 1.2) \times 10^{-3}\ m^{-1}$	$\mu = 0.14$ $k = 1.0 \times 10^{-3}\ m^{-1}$
Steel pipes incl. saddles for external tendons: with clean dry or lubricated strands[1]	$\mu = 0.20 - 0.30$ k = refer to 2 below	$\mu = 0.25$ k = refer to 2 below
Saddles for external tendons with internal HDPE tube over saddle: - bare strands - greased and plastic sheathed monostrands	$\mu = 0.12 - 0.15$ $\mu = 0.02 - 0.08$ k = refer to 2 below	$\mu = 0.14$ $\mu = 0.06$ k = refer to 2 below
Greased and plastic sheathed monostrands	$\mu = 0.04 - 0.07$ $k = (0.4 - 0.6) \times 10^{-3}\ m^{-1}$	$\mu = 0.05$ $k = 0.5 \times 10^{-3}\ m^{-1}$

1. μ-values depend on lubrication
2. The wobble factor can normally be neglected

3.2.2 Draw-in of wedge at lock-off: approx. 6 mm

This value is independent of the jack or tendon type. If necessary, e.g. for short tendons, compensation can be provided by appropriate procedures.

3.2.3 Other tendon force losses

In addition to friction and relaxation losses (see above), also concrete shrinkage and creep as well as a draw-in of the wedge during lock-of must be considered.

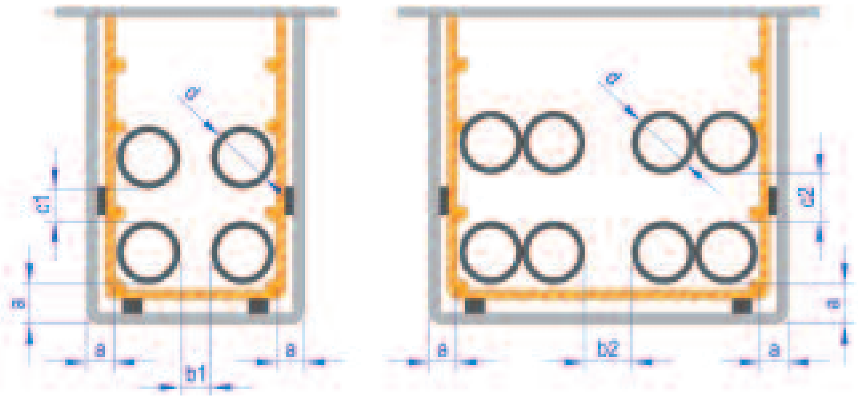
To calculate losses due to concrete shrinkage and creep, reference should be made to the technical documents and standards applicable to each project.

3.3 DUCT SPACING AND COVER

The cable layout patterns are dictated by the designer.

When detailing that cable layout, it is absolutely essential to consider the spacing of cables from one another, required cover, and radii of curvature. Usually the spacing and curvatures are laid down in standards, guidelines or national approvals. If not available, VSL recommends that the following guidance values be observed, these being minimum values:

Minimum spacing and cover of duct



Measurement (a)
- *Precast elements, elements protected from bad weather, soft environmental conditions* 30
- *In general* 40
- *Severe environmental conditions* 50

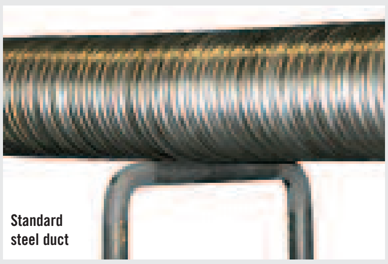
SPACING
b1, c1 = 0.7 times diameter of the duct
b2, c2 = 1.0 times diameter of the duct

3.4 SPACING OF THE SUPPORTS AND TOLERANCES

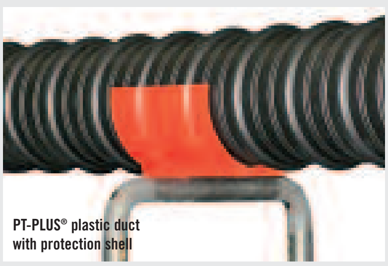
The spacing of the supports underneath steel and plastic ducts must be 10 to 12 times the internal diameter of the duct. Kinks are not permitted.

The fastening fittings must be sufficiently robust and close enough such that the ducts and tendons will not exhibit displacements or deformations in excess of the allowed tolerances. For tolerances on cable positions reference should be made to applicable standards and recommendations.

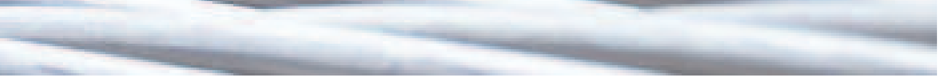
Moreover, under all circumstances and in every direction, whenever a cable displays or potentially displays deviation in the vicinity of an edge of concrete which could lead to spalling of concrete cover, an offset with respect to the theoretical axis is only tolerated provided that equilibrium reinforcing bars have been provided over this zone.


Standard steel duct

In determining minimum spacings and concrete cover requirements for ducts, reference should be made to applicable standards and recommendations, see 3.3.


PT-PLUS® plastic duct with protection shell

VSL Protection shells are recommended to be fixed on the duct at tendon supports for tendon radii R < 2 Rmin (see under 4.2), and where ducts risk to be dented by closely placed rebars.

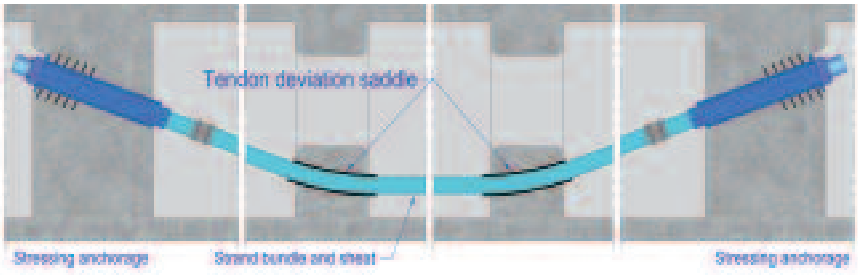


3.5 SADDLES FOR EXTERNAL TENDON

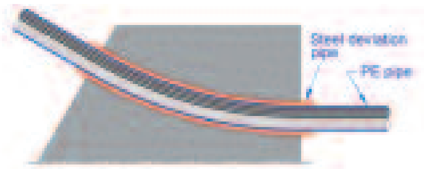
3.5.1 Saddles

Various solutions are used in practice. In most cases, saddles consist of a pre-bent steel tube cast into the surrounding concrete or attached to a steel structure by stiffening plates. The connection between the free tendon length and the saddle must be carefully detailed in order not to damage the prestressing steel by sharp angular deviations during stressing and in service. It is also important that the protective sheath be properly joined. If tendon replacement is a design requirement, the saddle arrangement must be chosen accordingly.

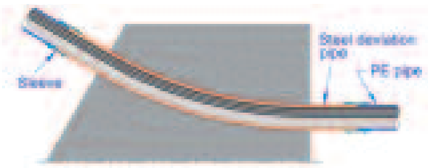
3.5.2 Various saddle arrangements



PE pipe through bell-mouth deviator, from anchorage to anchorage (most common detail)



PE pipe through deviator pipe, from anchorage to anchorage



Deviation pipe protrudes from concrete at sufficient distance and is coupled to PE pipe

3.5.3 Minimum radius of tendon curvature for external tendons

Tendon unit		Minimum
0.5"	0.6"	radius
up to 5 - 12	6 - 7	2.00 m
up to 5 - 19	6 - 12	2.50 m
up to 5 - 31	6 - 22	3.00 m
up to 5 - 43	6 - 31	3.50 m
up to 5 - 55	6 - 37	4.00 m
up to	6 - 43	4.50 m
up to	6 - 55	5.00 m

- The values are equivalent to approximately R_{min} (m) = (1.5 to 1.3) $\sqrt{F_{pk}\,[MN]}$ $\geq$ 2.0 m
- They apply to smooth steel and HDPE pipe and assume a straight length on either side of the deviation.

Diabolo bell-mouth for deviation points at diaphragm segment

4 - DESIGN REQUIREMENTS

4.1 ANCHORAGE ZONE REINFORCEMENT

The transfer of the prestressing forces from the anchorage into the concrete produces stresses which exceed the concrete strength and that must be withstood by special reinforcement. A distinction may be made between three types of reinforcement.

a) Local zone reinforcement in the immediate vicinity of the anchorage

For this purpose, spirals (helices) or appropriate orthogonal reinforcement are normally used. This reinforcement is considered as an integral component of the anchorage and its design lies within the field of responsibility of VSL. This reinforcement is specified in approvals and it may only be changed upon approval by VSL. The Anchorage data sheets show the required reinforcement for each anchorage.

b) General zone of reinforcement for resisting the spreading of forces in the structure

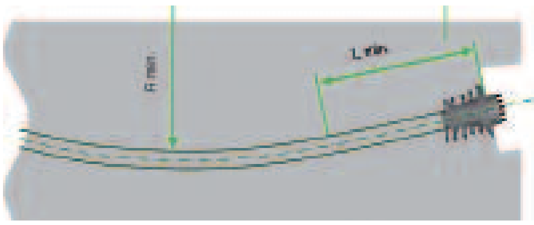
This reinforcement is designed by the project designer. Guidelines for its design can be found in VSL's report "Detailing for post-tensioning".

c) Reinforcement for spalling forces near stress free edges

4.2 MINIMUM RADIUS OF TENDON CURVATURE AND TANGENT LENGTH FOR INTERNAL TENDONS

Minimum radius of tendon curvature

Minimum tangent length



$R_{min\,(m)}$ = 3.0 x $\sqrt{F_{pk}\,[MN]}$ $\geq$ 2.5 m

L_{min} = 0.8 m for $F_{pk} \leq$ 2 MN

= 1.0 m for $F_{pk} \geq$ 2 MN, $\leq$ 7 MN

= 1.5 m for $F_{pk} \geq$ 7 MN

$R_{min\,(m)}$ = 2.50 m for unbonded tendons for 5-1 and 6-1

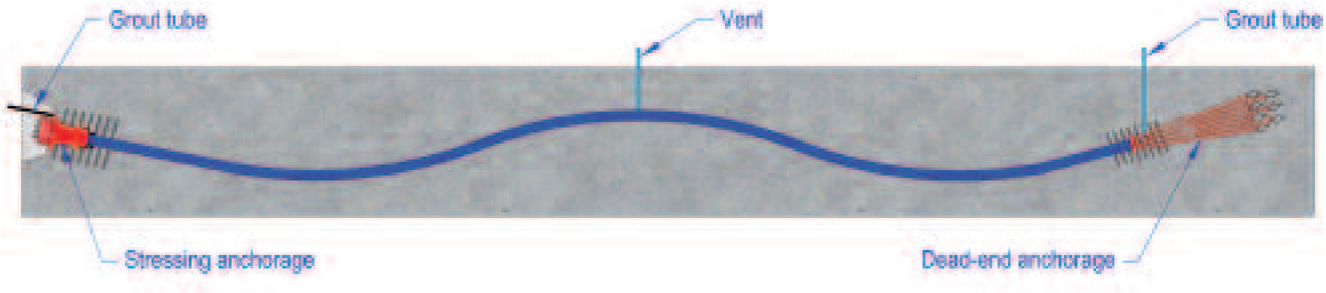
5 - INSTALLATION

5.1 ANCHORAGES

It is a requirement that the bearing plate / casting of anchorages are fixed perpendicular to the tendon axis.

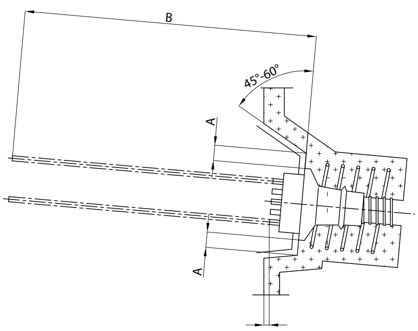
The block-out dimensions and clearance requirement as given under 5.3 should be followed. Departures from these data may be possible. Please contact VSL.

5.2 GROUT VENTS



Low point drains should only be foreseen where there is a risk of water freezing inside the duct and hence, drainage is required. As a general rule distance between grout connetions should not exceed 100 m. They should have a range of spacing between vents in the order of 30 – 70 m.

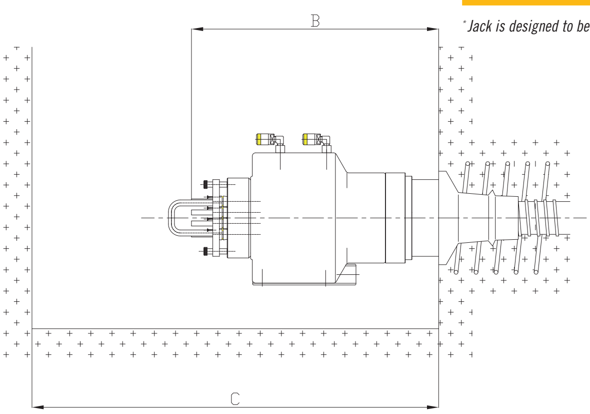
5.3 BLOCK-OUT DIMENSIONS AND CLEARANCE REQUIREMENTS

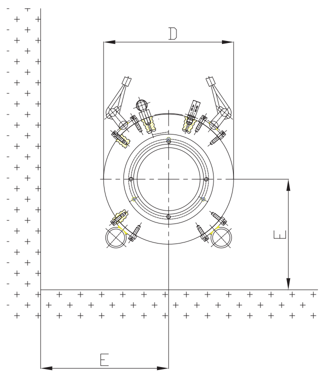


Concrete cover according to applicable standard

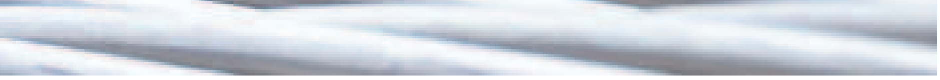
Jack type	A min.	B	C	D	E
ZPE-23FJ	–	300-360	1,200	116	90
ZPE-30	30	600	1,350	140	100
ZPE-3	30	500	1,000	200	150
ZPE-60	30	650	1,250	180	140
ZPE-7/A	30	650	1,400	300	200
ZPE-12/St2	50	520	1,100	310	200
ZPE-185*	50	620	1,220	300	180
ZPE-200	50	950	2,000	330	210
ZPE-19	50	700	1,500	390	250
ZPE-460/31	60	560	1,300	485	300
ZPE-500	80	950	2,000	585	330
ZPE-580*	80	860	1,620	500	280
ZPE-750	80	1,200	2,400	570	365
ZPE-980*	80	950	1,760	650	360
ZPE-1000	80	1,200	2,400	790	450
ZPE-1250	90	1,300	2,550	710	375
ZPE-1450*	90	1,010	1,850	770	420

Jack is designed to be used for 310kN UTS strands stressed to max. 85% of the 310kN. *Dimensions in mm*

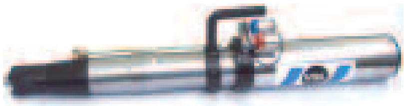




CREATING SOLUT



5.4 STRESSING JACK DATA



Type I (ZPE-23FJ)

Type II (ZPE-460/31)

Type III (ZPE-1000)

Designation	ZPE-23FJ	ZPE-30	ZPE-3	ZPE-60	ZPE-7/A	ZPE-12/St2	ZPE-185*	ZPE-200	ZPE-19
Type	I	II	III	III	III	II	II	III	II
Length (mm)	830	720	475	615	700	610	600	1,170	730
Diameter (mm)	116	140	200	180	280	310	295	315	390
Stroke (mm)	200	250	160	250	160	100	100	300	100
Piston area (cm^2)	47.10	58.32	103.6	126.4	203.6	309.4	309.3	325.7	500.3
Capacity (kN) (bar)	230 488	320 549	500 483	632 500	1,064 523	1,850 600	1,856 600	2,000 614	2,900 580
Weight (kg)	23	28	47	74	140	151	120	305	294
Used for 13mm/ 0.5” tendon types	5-1	5-1	5-2 5-3	5-2 to 5-4	5-6 5-7	5-12	5-7	5-12 5-19	5-18
Used for 15mm/ 0.6” tendon types	6-1	6-2	6-2	6-2 6-3	6-4	6-6 6-7	6-3 6-4 6-7	6-6 6-7	6-12

Designation	ZPE-460	ZPE-500	ZPE-580*	ZPE-750	ZPE-980*	ZPE-1000	ZPE-1250	ZPE-1450*
Type	II	III	II	II	II	III	II	II
Length (mm)	580	1,000	760	1,185	810	1,150	1,290	840
Diameter (mm)	485	550	500	520	645	790	620	765
Stroke (mm)	100	200	150	150	150	200	150	150
Piston area (cm^2)	804.0	894.6	961.7	1,247.0	1,652.3	1,809.5	2,168.0	2,436.9
Capacity (kN) (bar)	4,660 580	5,000 559	5,805 610	7,500 601	9,750 590	10,000 553	12,500 577	14,500 595
Weight (kg)	435	1,064	460	1,100	800	2,340	1,730	1,250
Used for 13mm/ 0.5” tendon types	5-22 5-31	5-22 5-31	5-12 to 5-31	5-31 to 5-55		5-37 to 5-55	5-37	
Used for 15mm/ 0.6” tendon types	6-18 6-19	6-18 to 6-22	6-12 6-19 6-22	6-31 to 6-43	6-27 6-31 6-37	6-31 to 6-37	6-43 to 6-55	6-43 6-48 6-55

Jack is designed to be used for 310kN UTS strands stressed to max. 85% of the 310kN.

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REDLINE

VSL SSI 2000 STAY CABLE SYSTEM

DESIGN
ENGINEERING
SUPPLY
INSTALLATION
MONITORING

VSL - LEADING IN STAY CABLE TECHNOLOGY

Industrial Ring Road – Bangkok, 2008

VSL – a worldwide network

From concept to site works, the VSL Network of locally operating units adds value throughout all stages of a project by providing fully-customised solutions, developed and implemented by highly-trained and experienced staff working in close partnership with clients. Customers have access to a local partner, while benefiting from global resources, know-how and expertise as well as VSL's continuing development of specialist construction techniques.

VSL – a commitment to quality, safety and sustainable development

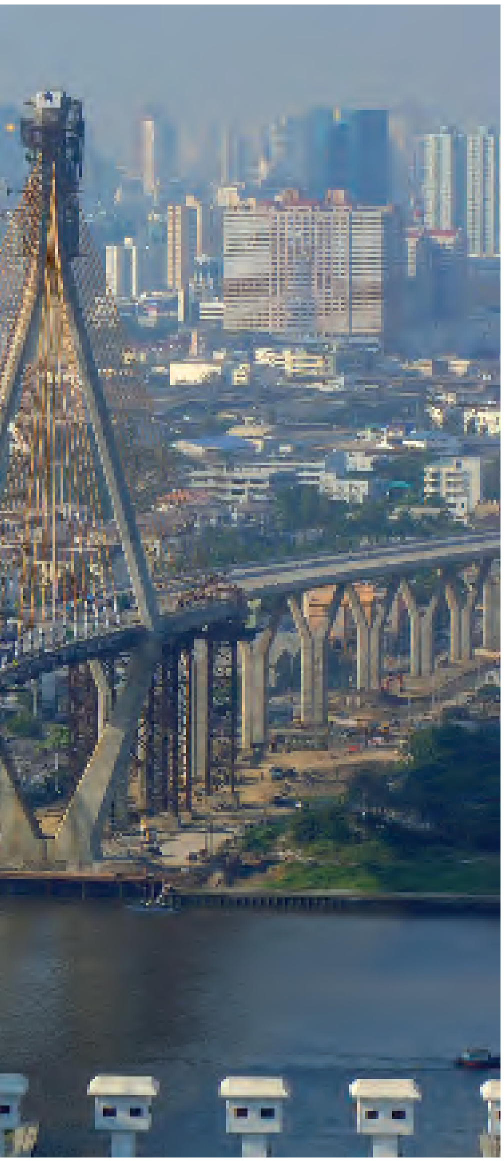
VSL pursues a strong quality, safety and sustainable development policy in keeping with its leading position as a specialist contractor. Proactive management systems have been established to address local needs while ensuring a high common standard throughout the company network.

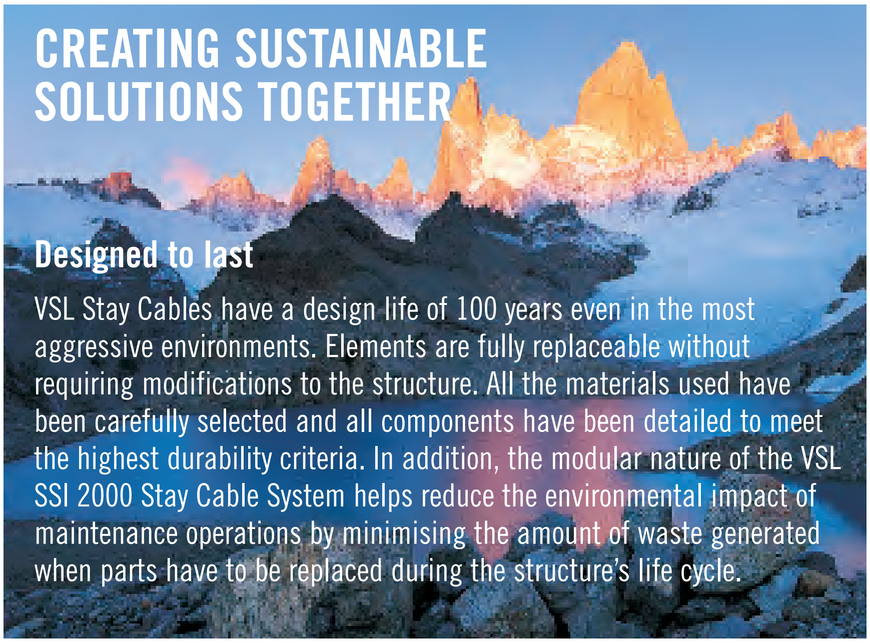
VSL recognises that its employees are the key to competitiveness, efficiency and safe working practices. The company is committed to "Safety First" and strives for "Zero Accident" by motivating and empowering its employees to act responsibly in order to achieve these goals.

VSL – a specialist stay cable contractor

As leader in stay cable technology, VSL offers the solutions to tackle today's challenges in cable-stayed construction and develops the next-generation systems in close collaboration with its clients. The recent boom in cable-stayed bridges with considerably increased spans and cable lengths calls for faster erection cycles and increases the dynamic demands on the stay cables. VSL's lightweight erection equipment, compact strand bundle solutions and its highly-efficient and reliable damping systems lead the way in meeting today's needs.

Its vast experience led VSL to launch the SSI 2000 system, which has been installed very successfully on more than 100 projects in recent years. VSL's latest developments extend the SSI 2000 range to provide even greater flexibility for a multitude of applications, while maintaining the system's proven outstanding performance. VSL's portfolio is now well over 150 cable-stayed bridges.





CREATING SUSTAINABLE SOLUTIONS TOGETHER

Designed to last

VSL Stay Cables have a design life of 100 years even in the most aggressive environments. Elements are fully replaceable without requiring modifications to the structure. All the materials used have been carefully selected and all components have been detailed to meet the highest durability criteria. In addition, the modular nature of the VSL SSI 2000 Stay Cable System helps reduce the environmental impact of maintenance operations by minimising the amount of waste generated when parts have to be replaced during the structure's life cycle.

New VSL developments in stay cable technology

SSI Saddle, a patented design facilitating simplified pylon layouts resulting in enhanced bridge aesthetics and increased structural efficiency

SSI 2000-C, a compact stay cable system with reduced cable diameter and therefore reduced wind drag

SSI 2000-D, a stay cable protected against corrosion by dehumidification techniques - a patented solution offering the smallest cable diameters available in strand technology and minimising wind drag while fully maintaining the advantages of strand-by-strand replacement

A choice of two damping systems to control cable vibrations efficiently, adapted to the characteristics of the structure

Modern engineering to stringent standards

Designers, owners and authorities are demanding:

- Increased long-term performance of stay cables, tensile members and anchorages; leak-tightness of the anchorage assembly; easy inspection and maintenance; the capability to replace cables with minimal interruption to bridge traffic; and reliable control of cable vibrations
- Minimal wind drag for long spans
- Outstanding static and fatigue behaviour, validated by performance testing
- Incorporation of damping systems at the time of installation or as part of dynamic retrofitting
- Improved aesthetics by using compact anchorages, saddles and coloured cables

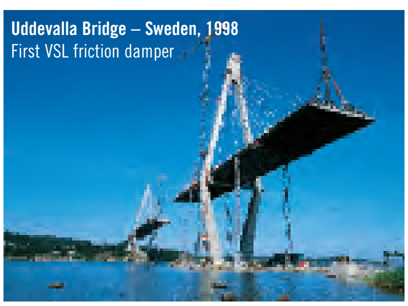
Main contractors seek:

- Simple interfaces between deck erection and stay cable installation with a reduced number of activities on the critical path
- Lightweight installation equipment, facilitating a flexible erection schedule that separates deck and pylon construction from the stay cable erection works and minimises the crane time required

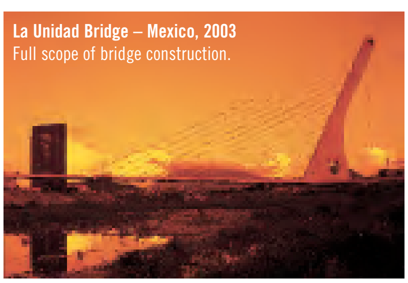
Owners benefit from:

- Enhanced durability
- Substantial savings on maintenance

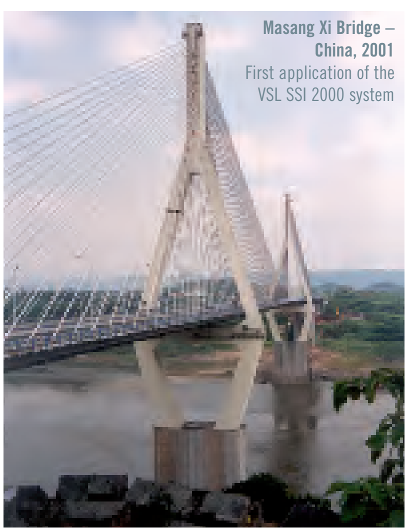
The VSL SSI 2000 Stay Cable System is designed to meet the requirements and applicable specifications issued by *fib* (International Federation for Structural Concrete), PTI (Post-Tensioning Institute) and CIP (Commission Interministérielle de la Précontrainte).



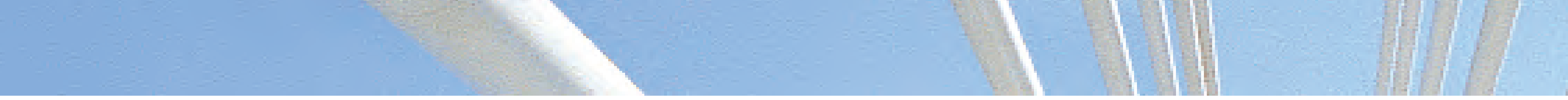
Uddevalla Bridge – Sweden, 1998
First VSL friction damper



La Unidad Bridge – Mexico, 2003
Full scope of bridge construction.



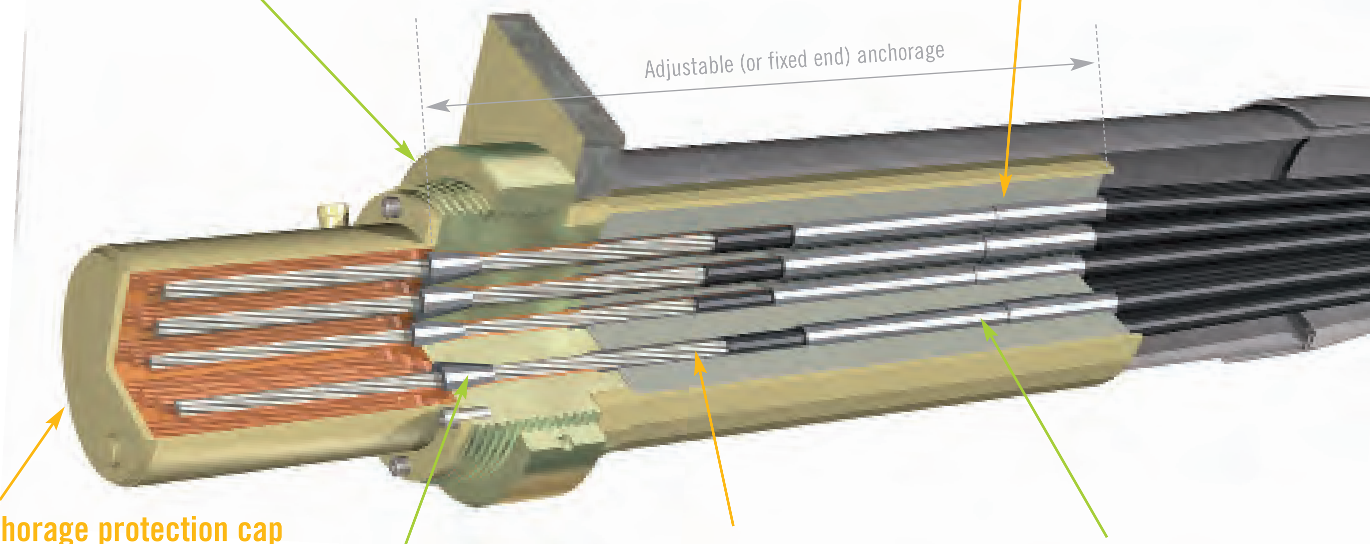
Masang Xi Bridge – China, 2001
First application of the VSL SSI 2000 system



SSI 2000: VSL STAY CABLE TECHNOLOGY

Compact anchorage
Fully prefabricated including its corrosion protection in controlled factory conditions

Several complementary barriers
For complete water tightness of the anchorage



Anchorage protection cap with flexible gel filler
Strands encapsulated by a polymerised and bonded filler, achieving reliable corrosion protection while allowing access for inspection if necessary

Replaceable strand system in a durable stay pipe
Sheathed, greased or waxed strands with optional galvanization, protected in an HDPE pipe with proven ageing performance. Each strand can be individually monitored, inspected and replaced

Individual encapsulation and deviation
Each strand is individually protected with a multi-layer barrier system inside a leak-tight anchorage assembly and is separately guided to filter bending stresses at the anchorage entrance

High fatigue resistance
Demonstrated in fatigue tests in accordance with *fib* and PTI requirements under combined tensile and bending action

The SSI 2000 Stay Cable System is based on VSL's proven strand technologies.

The SSI 2000 wedge anchorages and its tensile members as well as its protective system meet the most stringent requirements for durability, tensile capacity and fatigue performance. Its strand-by-strand technology ensures maximum flexibility and full capability for replacement.

High fatigue performance
The anchorage assembly is designed to control the deviation of individual strands and to filter cable vibrations outside the wedge anchorage zone. Its outstanding fatigue performance has been demonstrated in fatigue tests as specified in the latest recommendations by PTI and *fib* with imposed angular deviation of the anchorage from the cable axis. A tension ring or a guide deviator can be used to bundle the strands at the exit of the guide pipe.

Durability and multi-barrier protection
All SSI 2000 stay cables are engineered for a design life of 100 years in the most aggressive environments.
The unique feature of individual encapsulation of each strand within the anchorage assembly eliminates the risk of corrosion migration between strands.
The multi-barrier protection system is achieved in the free length by individually sheathed, greased or waxed strands with optional galvanization within the protective outer stay pipe. The protection is maintained in the anchorage assembly by a flexible gel filler injection, which has passed the stringent leak-tightness tests specified by PTI and *fib*.

Cable installation with lightweight equipment and minimum impact on other erection activities
The compact nature of the anchorages and the strand-by-strand installation with lightweight equipment frees tower crane time and does not require any heavy deck equipment. Therefore, the stay installation does not impair the key activities in a typical deck and pylon construction cycle.

GY

Low drag coefficient and aeroelastic stability

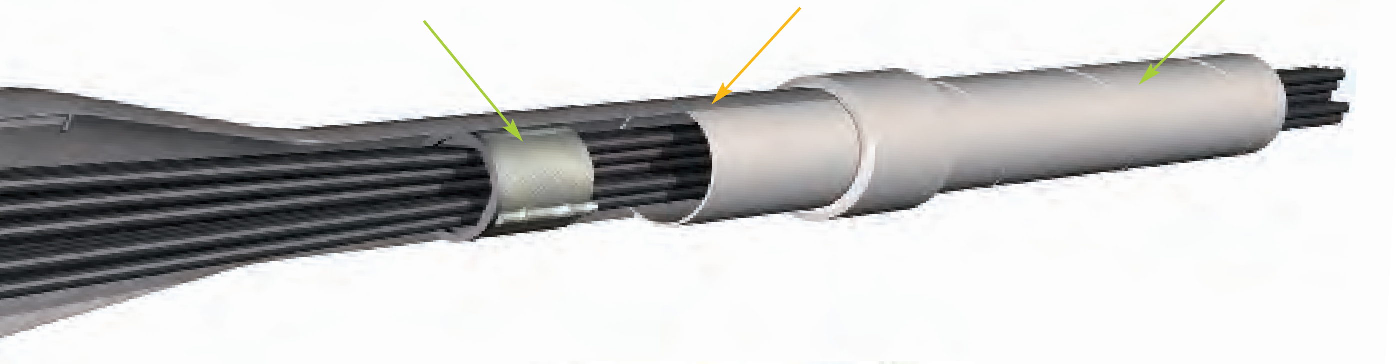
External helical ribs tested in wind tunnel for efficient control of rain-wind induced vibrations. Two options for even lower wind drag – SSI 2000-C and SSI 2000-D with reduced stay pipe diameters

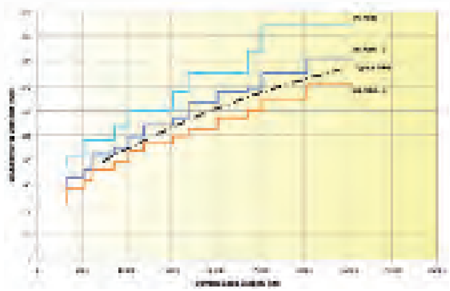
Free tension ring

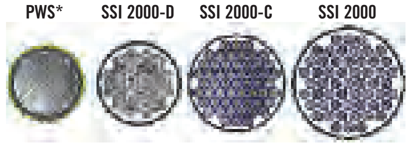
Located inside the stay pipe. Can be replaced by a guide deviator, depending on the geometry at the exit of the guide pipe

Anti-vandalism protection

Designed to protect the stay cable above deck level and to accommodate an optional damping system







Comparison of equivalent drag diameter of different types of stays
Equivalent Drag Diameter = O.D. Stay Ppipe x Drag Coefficient Cd
Cd = 0.6 for SSI 2000 has been determined in wind tunnel testing
Cd = 0.8 for PWS is based on typical project specification

** PWS = typical parallel wire system*

Cable replacement strand by strand with minimum traffic disruption

Strands can be individually monitored, inspected and replaced: entire cables can be replaced strand by strand. The use of lightweight equipment minimises the impact on vehicular traffic and cable replacement can be achieved under single lane closures.

VSL Dampers

The stay cable can be designed with two types of dampers, the VSL Friction damper or the VSL Gensui damper, or provision can be made for later installation.

SSI Saddle with fully replaceable strands

For extradosed bridges and cable-stayed bridges with compact pylon arrangements, VSL offers a patented saddle solution compatible with the SSI 2000 system. The compact saddle design allows for strand-by-strand installation and replacement and achieves a safe and reliable anchorage for unbalanced cable loads. Extensive fatigue testing has been carried out in accordance with *fib* requirements to demonstrate that there is equivalent performance between saddle and standard anchorages.

Three systems are available to meet project-specific aerodynamic requirements.

The standard SSI 2000 system with an optimised stay pipe to control rain-wind induced vibration and minimise wind drag

The stay pipe is fitted with a continuous helical rib, effectively suppressing rain and wind induced vibrations and reducing the wind drag on the cable. Extensive wind tunnel testing at speeds of up to 70m/s has been carried out for validation.

SSI 2000-C: the VSL Compact System for long cables

Reduced stay pipe diameters result in lower wind drag on the stay cable and hence a reduction in wind loads on the structure. This can be an important parameter in the design of long-span bridges. The SSI 2000-C compact stay cable range offers significantly reduced stay pipe diameters for the same permissible cable load. While this is the system of choice for exceptionally long cables, special tools are required for its installation.

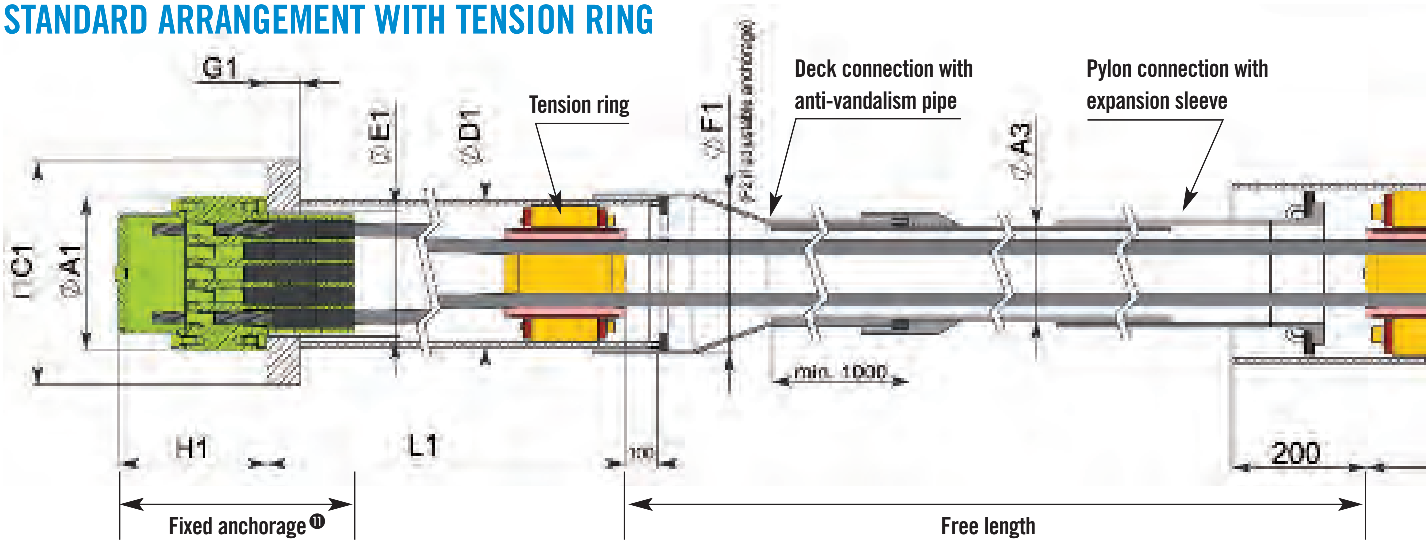
SSI 2000-D: the VSL Dehumidified System for even lower wind drag

The system maintains all the proven features of the standard anchorage system, while reducing further the cross section of the ducted strand bundle by eliminating the sheathing of the strands and providing equivalent corrosion protection through permanent dehumidification of the cable. The result is the most compact parallel strand stay cable on the market – a system with fully replaceable individual strands and unrivalled low wind drag.

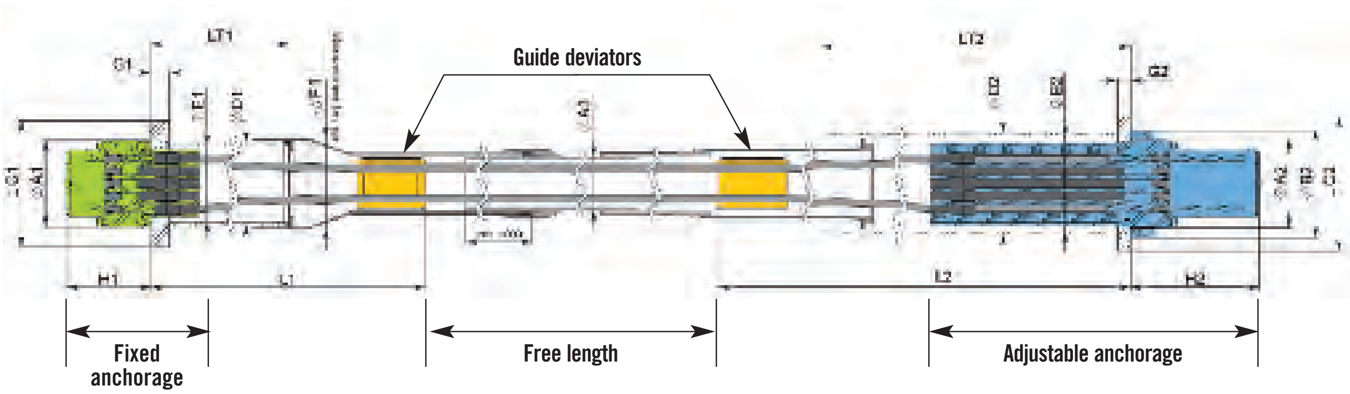
T I O N S   T O G E T H E R

VSL SSI 2000 MAIN DIMENSIONS

STANDARD ARRANGEMENT WITH TENSION RING

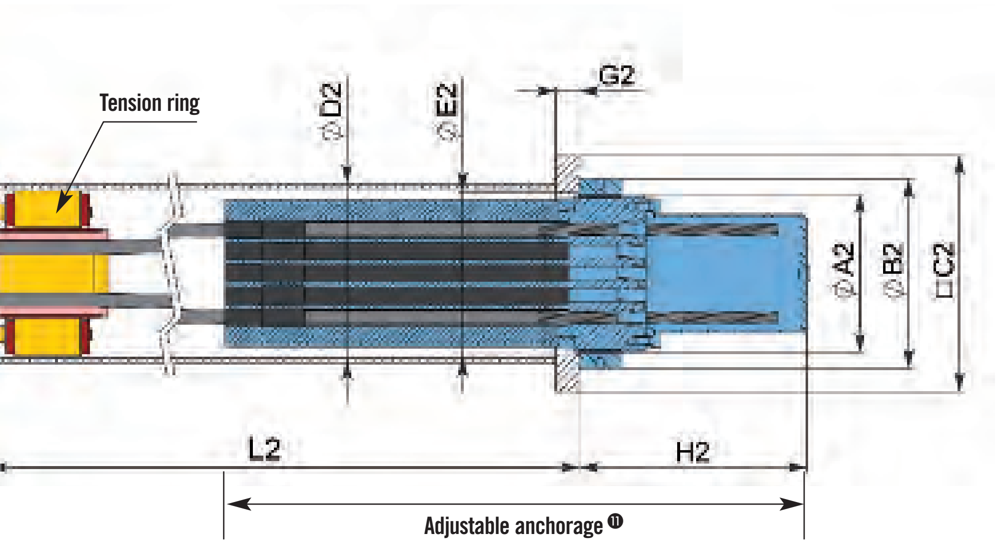


ALTERNATIVE ARRANGEMENT WITH GUIDE DEVIATOR



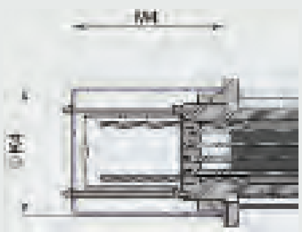
		STAY			STAY PIPE			FIXED ANCHORAGE						
CABLE UNIT (8)	NUMBER OF STRANDS	BREAKING LOAD AT 100% GUTS kN (1)	ADMISSIBLE LOAD AT 50% GUTS kN (2)	ADMISSIBLE LOAD AT 60% GUTS kN (3)	SSI 2000 mm (4)	SSI 2000-C ØA3/thk mm (4)	SSI 2000-D mm (5)	ØA1 mm	C1 mm (6)	ØD1/thk mm/mm	ØE1 mm	ØF1 mm	G1 mm	H1 mini mm
6-12	12	3,348	1,674	2,009	125/4.9	95/5.0	80/5.0	185	260	177.8/4.5	150	190	35	205
6-19	19	5,301	2,651	3,181	140/5.4	110/5.0	95/5.0	230	335	219.1/6.3	190	233	50	220
6-22	22	6,138	3,069	3,683	160/5.0	120/5.0	105/5.0	250	355	219.1/6.3	205	233	50	220
6-31	31	8,649	4,325	5,189	160/5.0	140/6.0	120/6.0	280	415	244.5/6.3	230	260	60	245
6-37	37	10,323	5,162	6,194	180/5.6	150/6.0	130/6.0	300	455	273/6,3	255	286	70	270
6-43	43	11,997	5,999	7,198	200/6.2	165/6.0	145/6.0	340	505	323.9/7.1	285	337	75	275
6-55	55	15,345	7,673	9,207	200/6.2	180/6.0	155/6.0	380	550	323.9/7.1	310	337	75	295
6-61	61	17,019	8,510	10,211	225/7.0	190/6.0	165/6.0	380	585	355.6/8	330	370	85	310
6-73	73	20,367	10,184	12,220	250/7.8	210/6.6	175/6.0	430	650	406.4/8.8	370	420	95	330
6-85	85	23,715	11,858	14,229	250/7.8	225/6.9	190/6.0	430	685	406.4/8.8	370	420	110	360
6-91	91	25,389	12,695	15,233	280/8.7	230/7.2	200/6.2	480	730	457/10	420	470	110	370
6-109	109	30,411	15,206	18,247	315/9.8	250/7.7	215/6.7	495	775	457/10	420	470	120	380
6-127	127	35,433	17,717	21,260	315/9.8	270/8.4	235/7.3	550	845	508/11	475	525	130	430
6-139	139	38,781	19,391	23,269	315/9.8	- (9)	- (9)	570	900	520/12	480	540	135	440
6-151	151	42,129	21,065	25,277	355/11.1	- (9)	- (9)	590	920	559/12.5	490	550	140	460
6-169	169	47,151	23,576	28,291	355/11.1	- (9)	- (9)	630	970	585/14	510	580	150	480
6-187[8]	187	52,173	26,087	31,304	400/12.3	- (9)	- (9)	660	1,000	600/15	550	620	160	490

(1) *Based on strand specification as per EN 10138 (150mm², 1860MPa); reduction required for ASTM A416 or BS 5896; GUTS = Guaranteed Ultimate Tensile Strength of strand*
(2) *Recommended max. service stress for stay cables as per fib bulletin No. 30 and CIP*
(3) *Recommended max. service stress for extradosed cables as per CIP*
(4) *Galvanized and sheathed strand with a minimum sheathing thickness of 1.5mm*

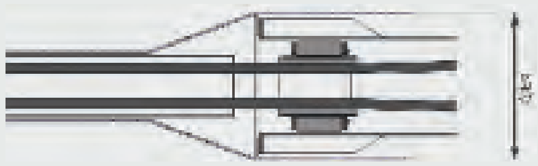


Adjustable anchorage [11]

OPTIONAL ITEMS

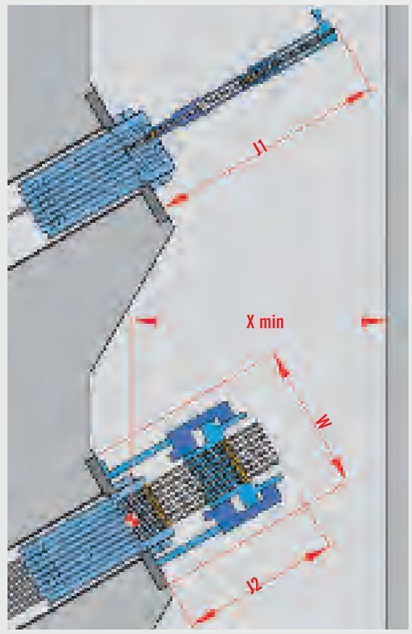


Optional anchorage cap
for adjustable anchorage in severe environments class C5-M and -I as per ISO 12944



Optional anti-vandalism pipe
for future provision of damper

Required clearances
In case of facing adjustable anchorages, it is recommended to provide two times the minimum clearance. If reduced clearances are required, please contact VSL.



Required jack clearances

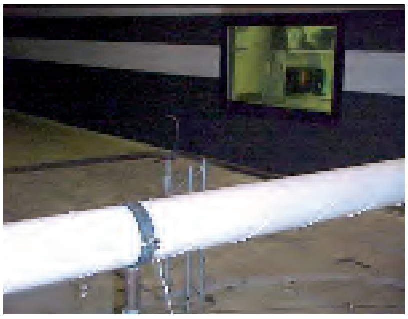
ANCHORAGE UNIT	W mm	J1 mm	J2 mm	Xmin mm
6-12 to 6-19	490	1,000	1,000	1,500
6-22 to 6-43	620	1,050	1,100	1,500
6-55 to 6-73	780	1,100	1,200	1,500
6-85 to 6-91	780	1,150	1,300	1,500
6-109 to 6-127	970	1,200	1,500	1,800
6-139 to 6-187	- [9]	1,250	- [9]	2,000

ADJUSTABLE ANCHORAGE								STANDARD ARRGT.		ALTERNATIVE ARRANGEMENT					OPTIONAL DETAILS		
ØA2 mm	B2 mm	C2 mm [6]	ØD2/thk mm/mm	E2 mm	ØF2 mm	G2 mm	H2 mini mm	L1 mm	L2 mm	LT1 DECK mm [7]	LT1 PYLON mm	LT2 DECK mm [7]	LT2 PYLON mm	HORIZONTAL FORCE ON GUIDE DEVIATOR kN [10]	ØF4 mm	ØK4 mm	M4 MINI mm
190	230	290	219.1/6.3	196	233	30	320	1,100	1,500	500	500	1,000	1,000	50	430	240	380
235	285	355	267/6.3	241	280	35	345	1,370	1,770	500	500	1,000	1,000	80	450	300	400
255	310	385	298.5/7.1	261	312	40	355	1,550	1,950	500	500	1,000	1,000	92	470	320	410
285	350	440	323.9/7.1	291	337	45	405	1,740	2,140	500	900	1,000	1,200	130	505	360	460
310	380	485	355.6/8	316	370	50	435	1,920	2,320	500	900	1,000	1,200	155	545	390	490
350	425	540	406.4/8.8	356	420	55	450	2,170	2,570	500	900	1,000	1,200	180	585	440	510
385	470	585	419/10	391	432	60	490	2,290	2,690	500	1,100	1,000	1,400	230	610	490	550
385	470	600	419/10	391	432	65	525	2,490	2,900	500	1,100	1,000	1,400	255	630	490	580
440	530	680	508/11	446	521	75	525	2,710	3,120	500	1,100	1,000	1,400	306	650	550	580
440	540	710	508/11	446	521	80	585	2,830	3,240	500	1,300	1,000	1,600	356	680	560	640
490	590	760	559/12.5	496	572	80	580	3,080	3,490	500	1,300	1,000	1,600	381	700	610	640
505	610	795	559/12.5	511	572	90	615	3,230	3,640	500	1,300	1,000	1,600	456	730	630	670
560	670	865	610/12.5	566	630	95	665	3,630	4,030	500	2,000	1,000	2,000	531	740	690	700
580	700	910	630/15	590	650	100	685	3,680	4,090	500	2,000	1,000	2,000	582	- [9]	- [9]	- [9]
590	720	940	640/15	600	660	100	695	3,770	4,170	500	2,000	1,000	2,000	632	- [9]	- [9]	- [9]
630	760	1,000	685/15	640	700	110	730	4,180	4,580	500	2,200	1,000	2,500	707	- [9]	- [9]	- [9]
660	800	1,050	720/15	670	740	120	770	4,190	4,590	500	2,200	1,000	2,500	783	- [9]	- [9]	- [9]

[5] *Galvanized strand in accordance with NF A 35-035*
[6] *Square bearing plate based on concrete strength of 45MPa cube (36MPa cylinder); dimensions can be adjusted for other concrete strength or steel structures*
[7] *Can be reduced if required; please contact VSL*
[8] *Larger units available on request*
[9] *Dimensions available on request*
[10] *SLS Level*
[11] *Fixed or adjustable anchorages are interchangeable between pylon and deck, see dimensions L1 and L2*

DURABLE DESIGN BACKED UP BY THOR

Durability and fatigue resistance are of utmost importance for stay cables, together with accessibility of components and monitoring of the structure.



Wind tunnel testing

Designed to last

VSL Stay Cables have a design life of 100 years in the most aggressive environments, as defined by the C4 and C5 categories of the ISO 12944 standard. All elements are fully replaceable without requiring modifications to the structure. All the materials used have been carefully selected and all components detailed to meet the highest durability criteria.
The SSI 2000 performance even exceeds the stringent durability requirements provided in the relevant recommendations from PTI and CIP. All non-accessible components are supplied with a factory-applied protection system ensuring a 100 year design life without maintenance. The accessible and replaceable elements are designed for a 25 year maintenance interval. The main tensile element of the cable, consisting of VSL-specified strands, is designed with a multi-layer protection system to match the performance of the anchorages. VSL provides a detailed maintenance schedule for each project and can assist clients in the implementation.

Leak-tightness testing for anchorages

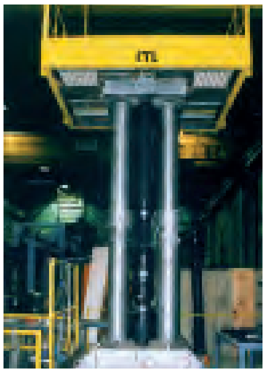
The anchorage is the most vulnerable part of a modern stay cable in terms of durability. In order to achieve a continuously protected transition from the free length of the cable into the anchorage zone, it is important to prevent water ingress through the component interfaces, particularly at the lower deck anchorage.
VSL has equipped its anchorages with a redundant multi-layer sealing system, which has passed the leak-tightness tests as defined by PTI and *fib*.

Accelerated ageing tests for coloured stay pipes

Correct material selection and manufacturing control is important to avoid variations in the characteristics of the HDPE components used for the outer protection layers of the free length. Ageing effects, UV radiation and/or pollution can deteriorate the condition of the stay pipe over time.
The evolution of the mechanical and colorimetric properties is verified by performing accelerated ageing tests.

With a focus on durability throughout all phases, VSL has developed specialist methods and procedures to ensure that the required installation quality is achieved and that any damage that might occur to the components during transportation, handling or installation is detected and rectified prior to project delivery. The modular nature of the system allows for simple replacement of any damaged parts without critical delay to the installation schedule.

Tensile test on an HDPE stay pipe sample after accelerated ageing



Fatigue test through 2 million cycles

Inspection of a 6-37 anchorage after a leak-tightness test. While being subjected to mechanical and environmental stresses, the anchorage is immersed in a dye solution with a pressure head of 3m.

ROUGH TESTING

Fatigue testing
The fatigue performance of the VSL Stay cable System has been demonstrated in many fatigue tests in accordance with PTI and fib recommendations with stress ranges of 200MPa at 45% GUTS upper stress over two million cycles and a 10mrad deviation at the anchorages.

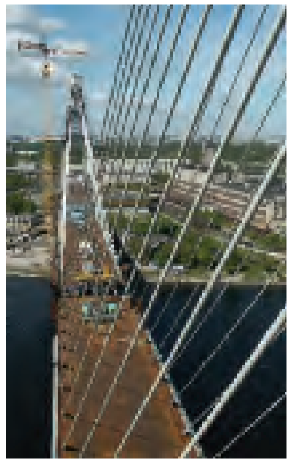
Proven technology for protecting the SSI 2000-D
With the SSI 2000-D system, VSL introduces proven dehumidification technology to stay cables, defining a new standard for the industry (see page 10). The D system maintains the highest standards of durability while eliminating the need for individual sheathing of the strands and hence allowing a significant reduction in cable diameters (see page 10).

Equivalent durability using an injected saddle with replaceable strands
The SSI Saddle is the first saddle in the market with injected but replaceable strands. The PE sheathing is removed from the strand at the saddle location. Each strand is individually guided through the saddle and bonded flexible gel filler, effectively preventing any oxygen or corrosive agents from reaching the strand. Full-scale fatigue tests have demonstrated that the saddle fulfils the same fatigue criteria as the standard anchorages and that no fretting corrosion occurs.

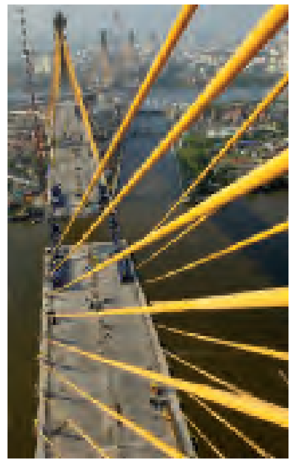
Durability requirements for stay cables

- The *fib* stay cable recommendations specify a design life of 100 years for stay systems installed in bridge structures and emphasise the need for adequate maintenance.
- The CIP stay cable recommendations propose a design life of 50 years for replaceable systems and 100 years for non-replaceable systems and require defined maintenance intervals

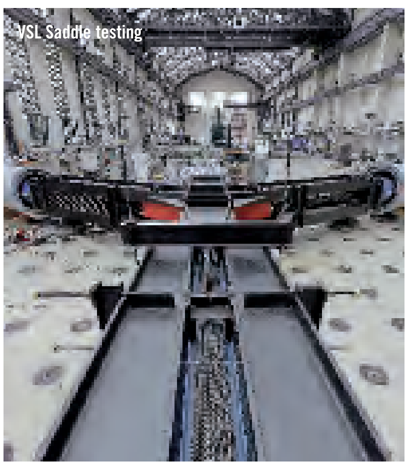
Red color stay pipes
(Sucharskiego Bridge, Poland)



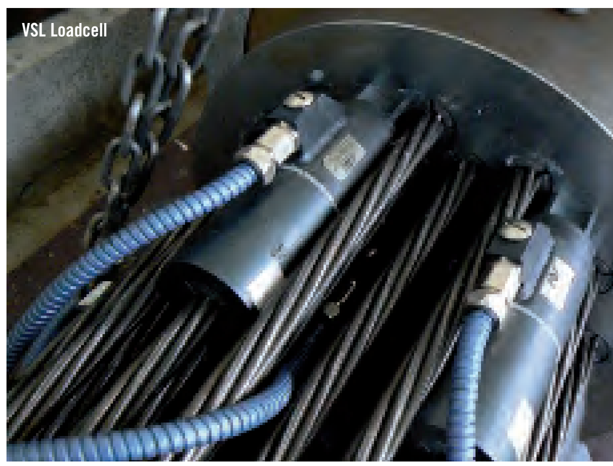
White color stay pipes
(Neva Bridge, Russia)



Gold color stay pipes
(Industrial Ring Road, Bangkok)



VSL Saddle testing



VSL Loadcell

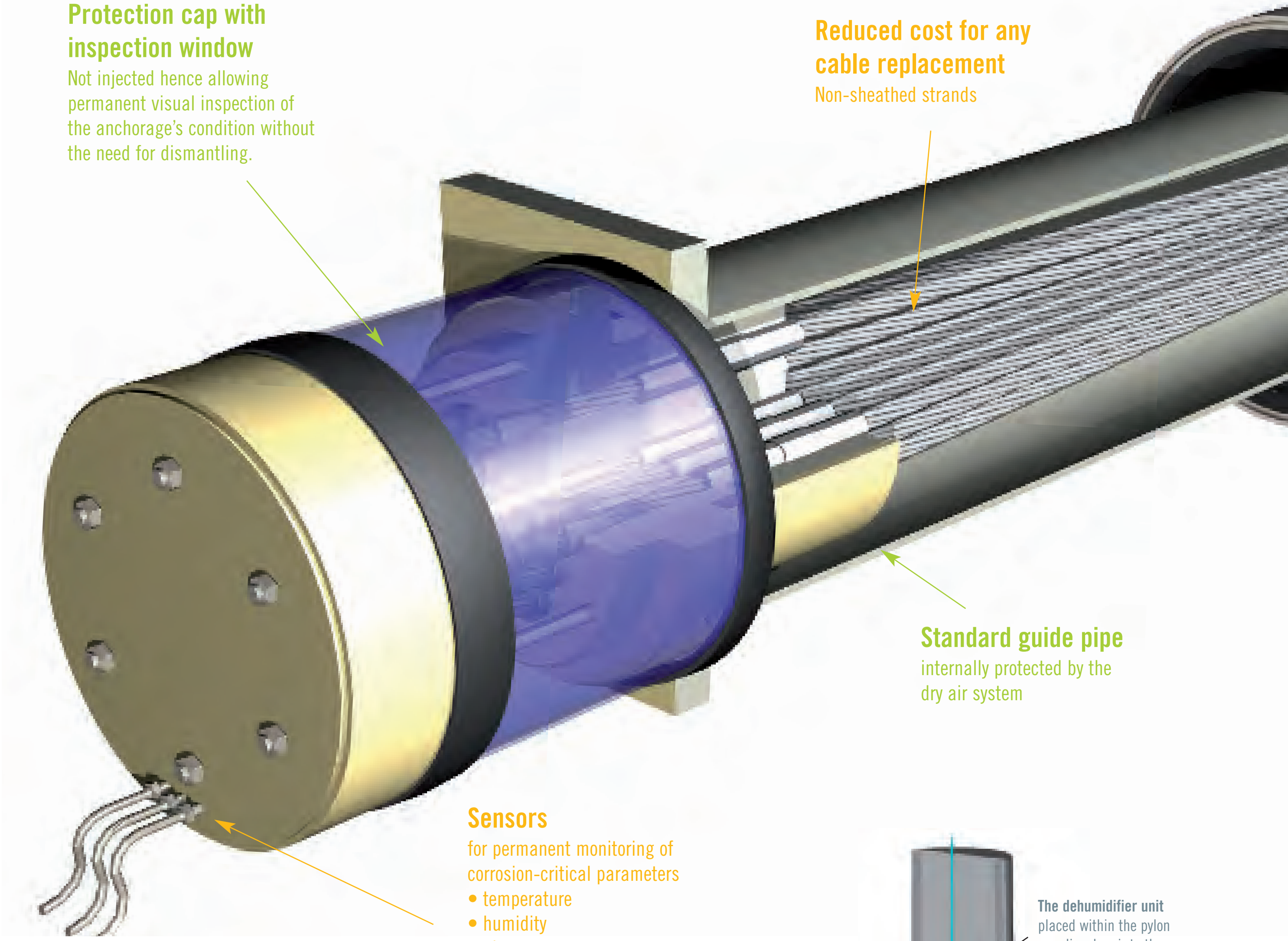
Enhanced durability by systematically controlling vibrations
VSL offers two types of dampers for effective control of the cable vibration: the VSL Friction Damper and the VSL Gensui Damper. Both dampers can either be installed during construction or retrofitted on existing structures. The dampers are designed for maximum durability by minimising the number of moveable parts and selecting the most durable materials.

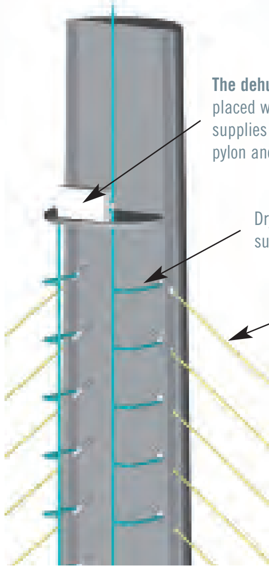
VSL Structural monitoring solutions
VSL also offers structural monitoring packages. Sensor solutions for permanent or temporary load and deformation measurements on cables can be combined with instrumentation of the structure. This allows collection of all the necessary data to optimise maintenance, validate design assumptions, diagnose mechanisms of deterioration and detect damage at an early stage.

TIONS TOGETHER

THE VSL SSI 2000-D SYSTEM patent pending

The world’s most compact stay cable strand protected by dry air

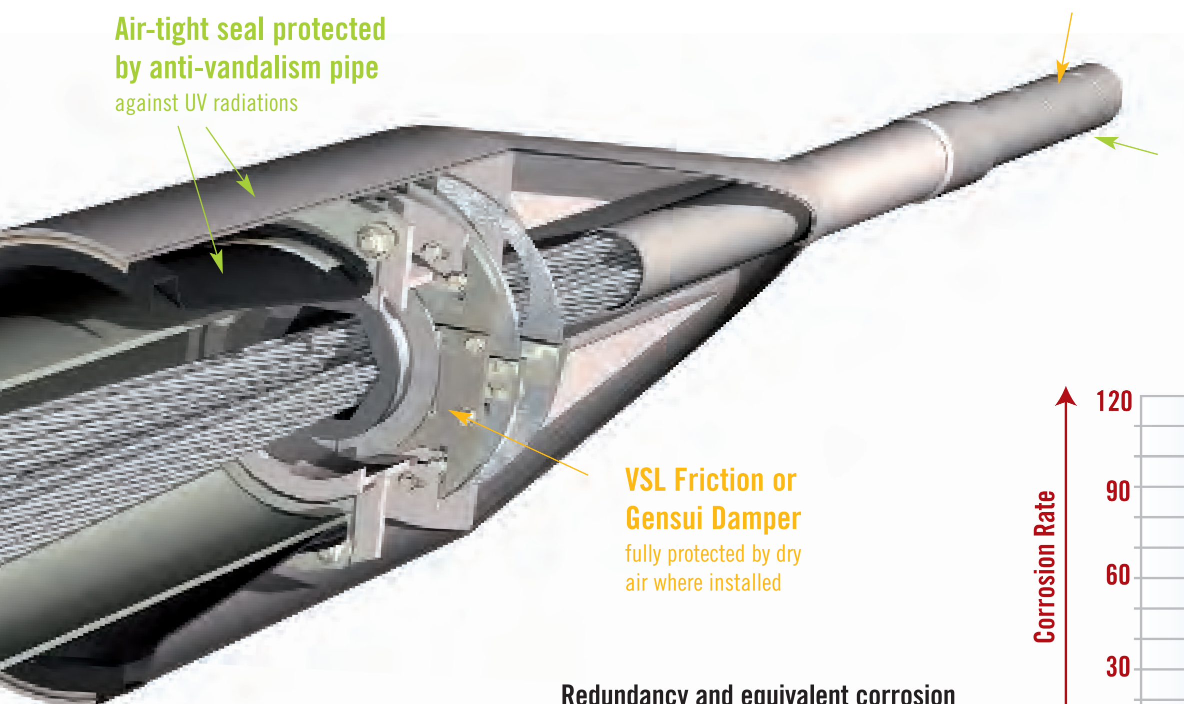


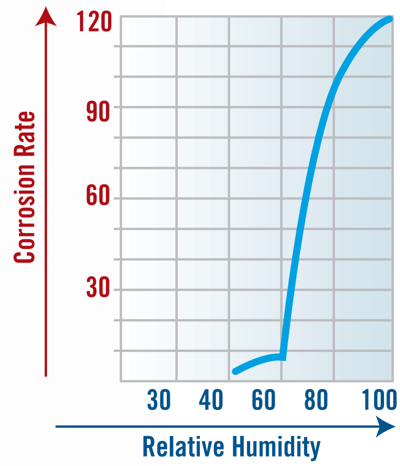


A compact bundle of unsheathed strands protected by dry air

With the SSI 2000-D system, VSL offers the most compact parallel strand stay cable in the market. The resulting wind drag is lower than that of parallel wire cables with equivalent capacity, while the system maintains all the typical SSI 2000 benefits when it comes to installation, inspection and replacement.

The galvanised, unsheathed strands are placed inside an air-tight enclosure, where any optional dampers can also be accommodated. A dehumidifier unit, typically placed inside the pylon, provides a constant supply of dry air at the pylon anchorages, while maintaining a permanent pressure differential between the inside and the outside of the cable. This prevents any ingress of moisture or other corrosive agents from the outside. All structural elements of the stay cable are





Iron Corrosion Rate at different Relative Humidities (%RH)

protected within this controlled environment where the humidity is maintained below the threshold that would trigger corrosion.

The applied pressure differential ensures that a leak in the system will only result in an increased air demand and will not jeopardise the protective mechanism.

Dry air – a reliable and proven solution

The concept of dehumidification systems to protect steel bridge decks and suspension cables was introduced in the 1970s and is today a well understood and highly reliable solution, applied to some of the most prestigious bridges around the world.

With the SSI 2000-D system, VSL has applied this proven technology to stay cables to offer a state-of-the-art corrosion protection solution.

Use of the latest dehumidification equipment keeps operational and maintenance costs extremely low. Running cost estimates can be provided on request.

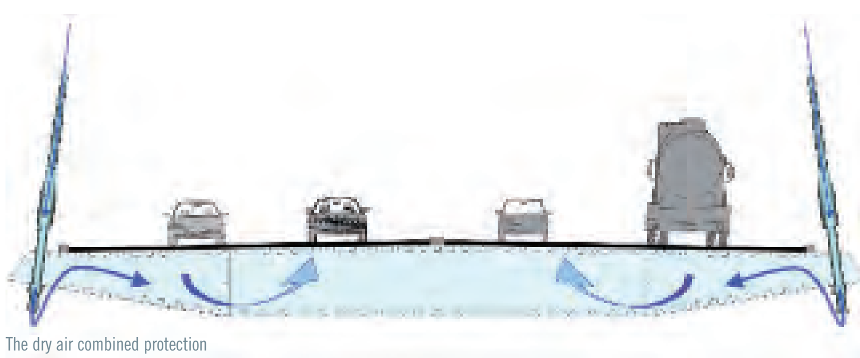
Redundancy and equivalent corrosion protection

The principle of multi-barrier protection remains unchanged even though the SSI 2000-D does not make use of individually sheathed strands:

- the stay pipe provides an airtight enclosure, protecting the tensile element against environmental effects
- a protective environment of dry air around the strands prevents moisture and corrosive agents from reaching the strands
- continuous galvanisation of the bare strand provides a final barrier against corrosion in case of scheduled removal or accidental loss of the other two barriers.

Permanent monitoring of the corrosion protection

The integrity of the corrosion protection system can be monitored permanently through continuous measurement of the corrosion-critical parameters of temperature and humidity inside the cable together with the air pressure. In addition, the protective caps at the anchorages can be fitted with transparent windows allowing a simple visual inspection of the anchorage condition. This significantly reduces the time and cost involved in periodic inspections of the anchorage components.


The dry air combined protection

Combining cable and deck protection

Where the SSI 2000-D is used with steel bridge decks or pylons with closed cross section, the dehumidification can protect both the stay cables and the structural steel elements by suitable sizing of the dehumidification units.

THE VSL SSI SADDLE

VSL has developed a new generation of stay cable saddles combining the advantages of injected saddles with full strand-by-strand installation and strand-by-strand replacement.

Saddles are the solution of choice for many bridges when it comes to compact and slender pylon designs or for extradosed structures. Replacing a pair of pylon anchorages with a single saddle simplifies the detailing and eliminates the need to anchor large splitting forces in the pylon.

The benefits of saddles are widely accepted but their general use had been prevented in the past by the issues of reduced fatigue performance compared to anchorage, the risk of fretting corrosion and the inability to replace single strands.

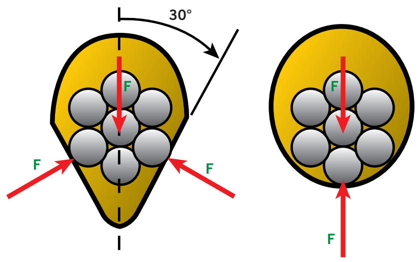
VSL responded to these challenges with the SSI Saddle. Individual guiding and encapsulation of the strands allows strand-by-strand installation, inspection and replacement while injecting the guide tubes with a special, polymerised and bonded flexible gel filler prevents any ingress of oxygen, hence eliminating the risk of fretting corrosion. The result is a saddle with fully replaceable strands that achieves the same fatigue performance as standard SSI 2000 Anchorages.

The V-effect; maximum friction using wedge action

The SSI Saddle is a steel box filled with Ductal® ultra-high-performance concrete and featuring V-shaped guide voids for each individual strand. This patented geometry provides an efficient wedge action, continuously gripping the strand by friction along the deviated length, while minimising fretting under cyclic loading. The entire saddle is detailed such that the deviation occurs entirely in the strand-to-Ductal® interface with no intermediate layers that could deteriorate over time.

Independently guided and replaceable strands

The saddle allows unrivalled single strand installation, inspection and replacement. Strands can be individually stressed and de-stressed. In the same way that larger anchorage units can have spare strand positions, the saddle can also incorporate additional guide voids to give the option for a future increase in cable capacity.



SSI Saddle section, with the "V" shaped strand hole compared to circular guide void

Seamless integration with the SSI System

The Saddle uses the same strand as the standard SSI 2000 System, with no additional treatment required. The removal of the tightly extruded PE-coating on the deviated length inside the saddle is performed on site.

Continuous multi-barrier corrosion protection

The PE-coating of the strands is removed inside the deviated length of the saddle to achieve strand-to-Ductal® contact. As with the other SSI components, multi-barrier protection has been incorporated:

- An outer casing consisting of a steel box and Ductal® gives protection against ingress of water and corrosive agents
- Injection of the guide voids with a polymerised, bonded, flexible gel filler gives a reliable seal against moisture and oxygen
- Galvanisation of the strand provides protection during the installation period prior to the injection

Transfer of high differential cable forces into the pylon

Unbalanced loading between bridge spans results in a need to transfer differential cable forces to the pylon, which is achieved by high friction in the V-shaped guide voids in the saddle. Consistent friction coefficients in excess of 0.4 are attained in tests.

VSL Saddle design with cable connection

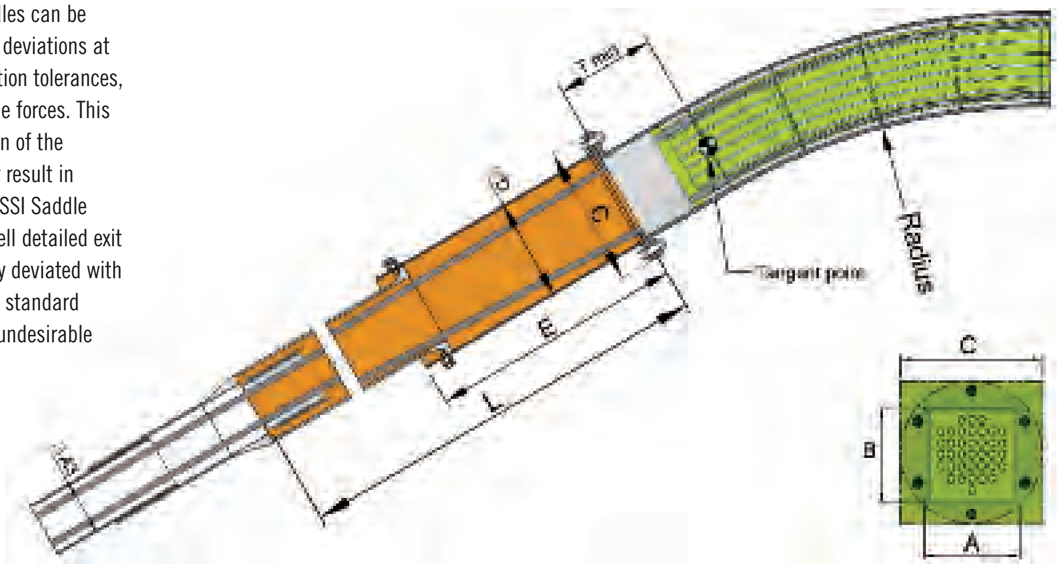
Proven fatigue performance

VSL has carried out extensive fatigue testing with the SSI 2000 Saddle in accordance with *fib* recommendations, including a full scale test using a 6-55 unit.

Controlled filtering of angular deviations at the saddle exit

Stay cable anchorages and saddles can be subjected to significant angular deviations at their exits as a result of installation tolerances, vibrations and variations in cable forces. This could cause gradual deterioration of the protective layers of the strand or result in premature fatigue damage. The SSI Saddle avoids this by incorporating a well detailed exit where each strand is individually deviated with the same characteristics as in a standard anchorage, without introducing undesirable stress into the pylon surface.

Fatigue testing of the VSL Saddle
2,000,000 cycles successfully completed



Main dimensions (using VSL SSI 2000)

CABLE UNIT	STAY			STAY PIPE CONNECTION				SADDLE BODY			
	NUMBER OF STRANDS	BREAKING LOAD AT 100% GUTS kN	MAX. RADIAL BEARING STRESS AT 2.4m RADIUS MPa	L mm	E mm	ØD mm	ØA3/thk. mm	C mm	B mm	A mm	T min mm
6 - 12	12	3,348	9	1,160	250	193.7 / 4.5	125 / 4.9	320	200	185	300
6 - 19	19	5,301	11	1,760	250	244.5 / 5	140 / 5.4	370	235	230	300
6 - 22	22	6,138	13	1,900	500	244.5 / 5	160 / 6.2	370	235	230	300
6 - 31	31	8,649	13	2,220	500	273 / 5	160 / 6.2	400	275	300	300
6 - 37	37	10,323	15	2,580	500	323.9 / 5.6	180 / 5.6	450	310	310	300
6 - 43	43	11,997	14	2,540	500	355.6 / 5.6	200 / 6.2	500	310	375	300
6 - 55	55	15,345	18	2,880	800	368 / 6	200 / 6.2	510	350	380	300
6 - 61	61	17,019	20	3,240	800	406 / 6.3	225 / 7	550	395	410	300
6 - 73	73	20,367	20	3,470	800	430 / 8	250 / 7.8	570	395	470	300
6 - 85	85	23,715	23	3,590	800	450 / 8	250 / 7.8	590	430	480	300
6 - 91	91	25,389	23	4,010	800	480 / 8	280 / 8.7	620	470	520	300
6 - 109	109	30,411	24	4,120	800	500 / 8	315 / 9.8	640	470	580	300
6 - 127	127	35,433	26	4,700	800	560 / 8	315 / 9.8	700	545	580	300

VSL SOLUTIONS FOR CABLE VIBRATION

VSL provides expertise to assist owners and designers in analysing the risks of cable vibrations and proposes appropriate mitigation measures.

The mechanisms of dynamic excitation of stay cables are complex and can only be partially addressed in the various general recommendations for cable-stayed structures. VSL applies various stability criteria to estimate the risks of unacceptable cable vibration in order to determine the structural and additional damping requirements.
The VSL Stay cable system offers a modular approach for mitigating the risks of cable vibrations.

Helical ribs

The outer stay pipe is fabricated with double helical ribs, which have been optimised in wind tunnel tests for maximum efficiency and minimum drag against vibrations induced by rain and wind.

Two different types of dampers

VSL offers two damping solutions for stay cables: the VSL Friction Damper and the VSL Gensui Damper. Both are highly efficient as well as being extremely durable and require little maintenance. The outstanding long-term performance is based on minimising the number of movable parts, which reduces the wear and tear. This approach makes VSL Dampers significantly more robust than other damping systems.

The VSL Friction Damper is a highly efficient and durable damper for more critical applications, such as long cables or structures with an increased risk of vibration.

VSL Friction Damper

VSL Friction Damper – high performance for critical cases

High efficiency: Several comparative tests on full-scale cables fitted with dampers have demonstrated the exceptional efficiency of the VSL Friction Damper. The measured performance has repeatedly exceeded the specified requirements.

Outstanding durability: The friction damper achieves an excellent long-term performance by being designed to work only when needed. It comes into use once the displacement reaches a level that is considered critical for the cable's performance. Once activated, the damper achieves its maximum damping effect immediately.

Aesthetic solution: The installation of damping systems on stay cables has to be carried out with minimum impact on the visual appearance of the structure. The addition of external damper supports is often undesirable and the compact nature of the friction damper allows it to be fully integrated into the anti-vandalism pipe of the SSI 2000 System.

Other benefits of the VSL Friction Damper:
- Easy access for simplified inspection and maintenance
- Tuning of the damping performance by adjustment of the friction force without the need to dismantle the damper
- Retrofitting on any existing stay cable (strand or parallel wire) on structures where unexpected cable vibrations have been observed
- All components can be replaced on site
- Damping characteristics independent of temperature variations or vibration frequencies

VSL Gensui Damper - Tailor-made efficiency for short and medium stay cables

High-damping rubber: The VSL Gensui Damper is composed of several special rubber pads and the cable's dynamic energy is dissipated by shear deformation. The damper pads are made of a high-damping rubber developed and manufactured by Sumitomo Rubber Group.

Great simplicity: The damper is modular, with the number of pads required depending on the dynamic characteristics of the cable. The simple and versatile system can be easily adapted to any cable size whether as a new installation or as part of a retrofitting solution.

Excellent durability: The high-damping rubber pads have a long design life and a high fatigue resistance. They require only minimal maintenance, which allows dampers to be installed even at the pylon where maintenance access is difficult to provide.

Tailor-made performance: The damper performance depends purely on the damping characteristics of the rubber pads and can be adjusted by varying the number and type of pads used. The damper performs at its best on short to medium length cables. Its performance can be further enhanced by increasing the distance between the damper and anchorage or by installing a second damper at the pylon. Its performance is independent of the vibration mode and is not particularly sensitive to temperature variations and frequency.

Compact aesthetics: The Gensui damper can be fully integrated into the SSI 2000 anti-vandalism pipe with minimum impact on the cable aesthetics. Solutions for retrofitting include simple neoprene boots or compact mounting frames in the case of external dampers.

The VSL Gensui Damper is a simple and robust damper optimised for short to medium length stay cables with moderate damping requirements and for extradosed cables. Both dampers are incorporated into the SSI 2000 system as internal dampers, fully protected inside the anti-vandalism pipe. The dampers can either be installed together with the cable or retrofitted.

Connection detail for subsequent installation of a VSL damping system
The flange welded at the end of the guide pipe and the increased diameter of the anti-vandalism pipe are designed to allow for later installation of a VSL damping system, should cable vibrations need to be rectified. While there is no damper installed the cable is equipped with a guide deviator or with a tension ring.

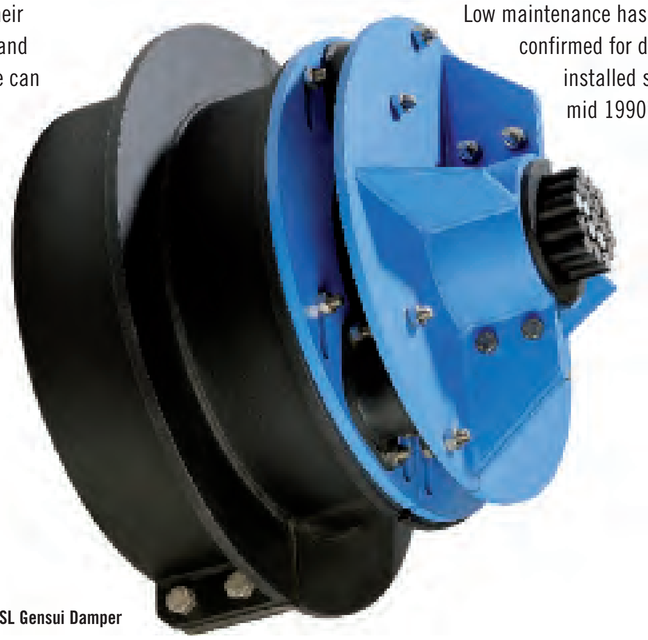
Cable equipped with the VSL Friction Damper
In the event of unexpected cable vibrations, a choice of VSL damping systems - Friction Damper or Gensui Damper - can be installed without modification to the cable assembly.

Provision of stabilising cross-ties

While VSL recommends the use of dampers for efficient vibration control, the SSI 2000 System allows also for installation of cross-ties if requested. Cross-ties help to increase the critical wind speed for aeroelastic instability by increasing the natural frequency of the cable. They are however only efficient in the cable plane and their installation and maintenance can constitute a significant additional cost.

Tried and tested

VSL dampers have demonstrated their outstanding performance in a series of comparative tests on full-scale samples conducted by organisations such as Shanghai's Tongji University, Hong Kong Highways Department and the Korea Expressway Corporation, as well as on sites. Low maintenance has been confirmed for dampers installed since the mid 1990's.



VSL Gensui Damper

STRAND BY STRAND METHOD FOR INSTA OR REPLACEMENT OF CABLES

The strand-by-strand installation methods developed by VSL offer maximum flexibility and can be adapted to specific needs.

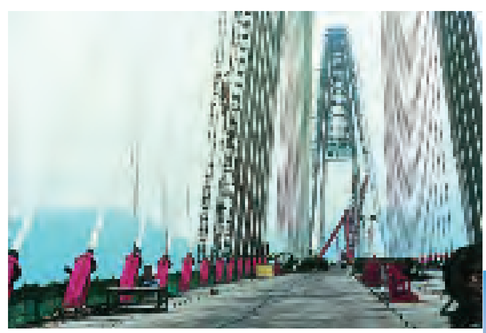
All SSI 2000 cables are installed strand by strand using extremely compact equipment and can be inspected and replaced if necessary in the same manner.

An optimised solution to streamline complex bridge erection cycles

The equipment can be handled manually at the anchorage location inside or outside the pylons, whatever their shapes. The strand reels are light and compact compared with prefabricated cables and can be easily lifted, transported and handled. This renders the cable installation largely independent of the logistics of deck and pylon construction. As a significant part of the installation can be carried out off the critical path, tower crane usage is reduced, resulting in cost and programme savings.

The preferred option for cable inspection or replacement under traffic

The compact equipment allows for inspection and replacement of the entire stay cable with minimum impact on the bridge traffic, as a single lane closure is typically sufficient to provide a safe working space. In addition, the strand-by-strand replacement makes the loss of cable force during the works negligible, allowing unrestricted vehicle movements.



Replacement of a 300m cable
In May 2002, VSL replaced a 298m-long cable on the Ching Chau Min Jiang Bridge, which had been damaged when a barge crane collided with the bridge during a typhoon. Site conditions did not allow access for a mobile crane and so all the equipment had to be handled manually. The cable was replaced strand by strand with VSL's lightweight equipment. This operation demonstrated that even long cables can be replaced with minimum disruption to bridge operations.

TALLATION

The main advantages of the VSL strand-by-strand system installation

- Absolute flexibility to adjust the cable length during construction to address variations in the bridge geometry or late changes to the deck erection methodology
- No requirement for provision of off-site prefabrication facilities
- No significant additional construction loads on the partially-completed structure as light and compact equipment is used.
- Fast erection cycle with partial and staged installation of cables for light composite deck assembly
- No additional requirement on the project's critical path for the use of tower and deck cranes during cable erection, thus reducing the risk of delay
- Lightweight shop prefabricated anchorages can be pre-installed during deck and pylon construction
- Easy second stage stressing with monostrand jacks, providing greater flexibility to designers and contractors by avoiding the relocation of heavy stressing and access equipment
- Improved site safety due to reduced component weights and simplified access arrangements
- Full strand-by-strand replacement
- Fully compatible with the VSL Saddle

A matching saddle

The SSI Saddle has been specifically designed to allow application of the same strand-by-strand principles for installation, inspection and replacement of the stay cables. All strands are individually encapsulated and guided within the saddle assembly, which combines the advantages of a saddle solution with the benefits of a strand system.

VSL lifting winch

Specialist equipment and procedures

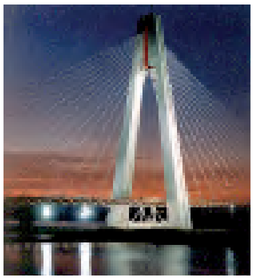
The continuous stay pipe is welded on site from elements of standard length. The strands are delivered to site in compact coils and are installed one by one using a small winch system. They are individually stressed by a lightweight monostrand jack from either the deck or the pylon end. The VSL AMS system provides fully automatic control, recording and data management for the stressing operation on site. Specialist procedures are implemented to ensure an equal final force in all strands and safe anchoring of low cable forces at intermediate stressing stages. The system provides absolute freedom to engineers to specify stressing either to a cable force or cable length, depending on the characteristics of the structure. It is even possible to change the cable length during construction if required. Final tuning of the completed cables can be carried out either by monostrand jack or by compact multistrand jacks. A ring nut is provided on the stressing anchorage to allow a reduction in the cable force if necessary without re-gripping the strand.

VSL Automatic Monostrand Stressing (AMS) system
The AMS system provides fully-automatic control and recording of the stressing operation and management of all relevant stressing data. The system allows absolute flexibility in defining the stressing parameters to achieve either a required cable force or cable elongation

Flexible erection cycles
Lightweight equipment with independence from heavy craneage allows simultaneous working and a reduced number of activities on the critical path. The cable installation activities can be easily adapted to match the requirements of the deck construction cycle.

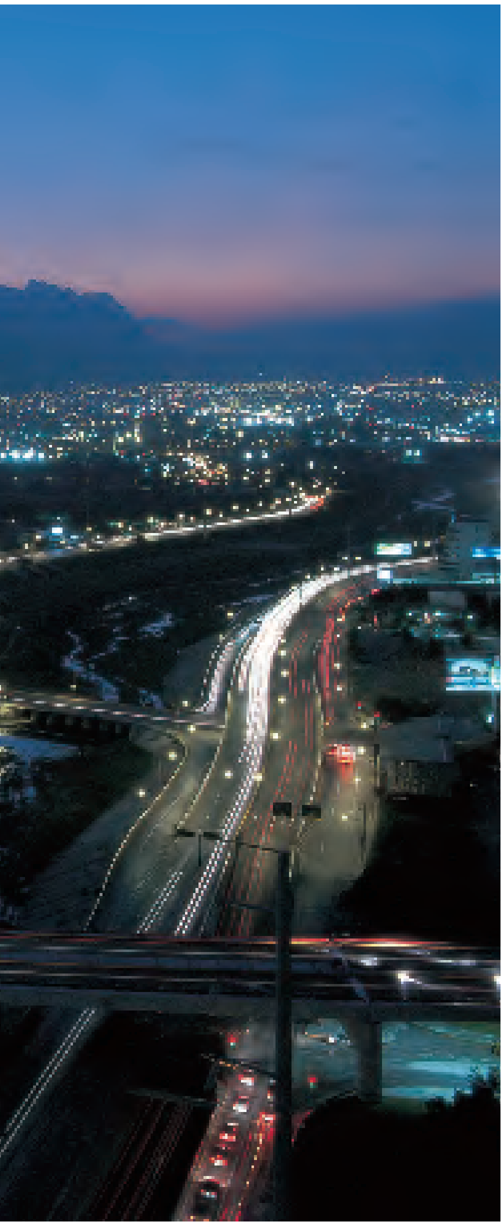
T I O N S   T O G

Puente de la Unidad Bridge, Mexico (2003)
In a 50/50 JV, VSL provided project management, complete technical and method support and part of the production management. VSL Mexico also supplied and installed the post-tensioning, and the SSI 2000 Stay Cable System



Badajoz Bridge, Spain (1994)
Cables equipped with friction dampers

Ponte Europa Bridge, Portugal (2002)
186m main span length - 91 strand cables

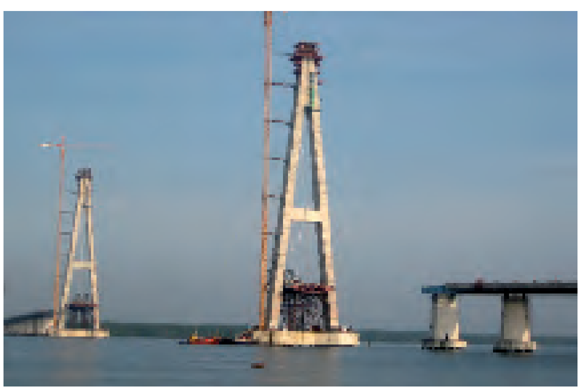


Batam Tonton Bridge, Indonesia (1997)
Package: design, supply and installation of stay cables, deck form-travellers and pylon formwork. Construction engineering for the superstructure construction

Lazarevskyi bridge, Saint Petersburg, Russia (2008)
Stays ranging from 6-55 to 6-73

Taipei Ring road Bridge, Taiwan (2009)
13 pairs of stays on each side of the pylon

Papendorpse Bridge, Netherlands (2002)
120 strand cables

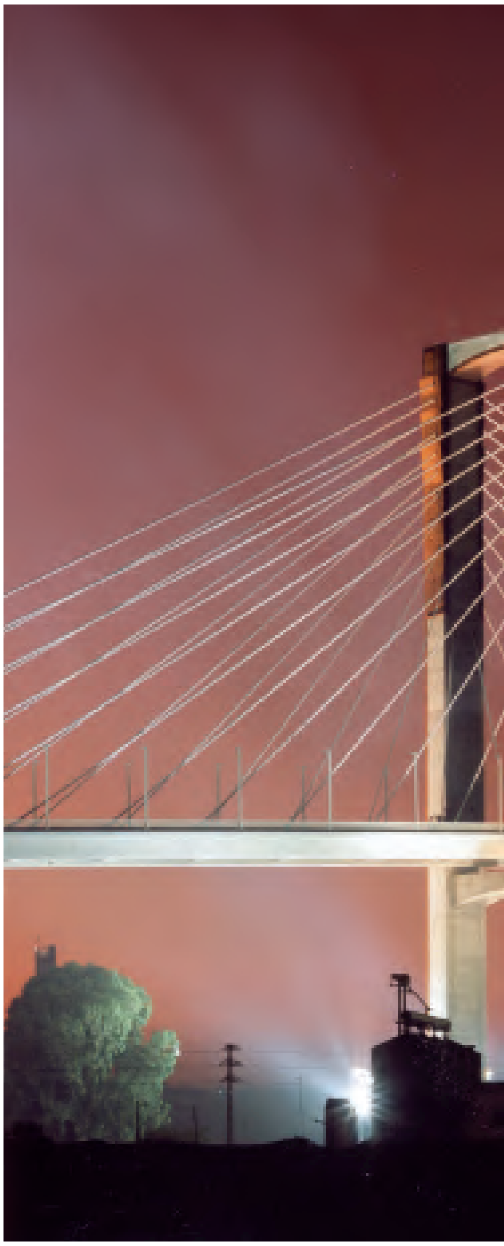


Sungai Johor Bridge, Malaysia (2008)
85 strand cables with length up to 275m

T I O N S T O G E T H E R

Sucharskiego Bridge, Poland (2001)
Supply and installation of stay cables,
with VSL Friction Dampers

Centenario Bridge, Spain (1991)
552m bridge length, with 264m for the main span



Wadi Abdoun Bridge, Jordan (2006)
Curved deck with inclined pylon

Yichong Yiling Bridge, China (2001)
Supply of stay cable system, erection equipment, stay cable engineering, site management and site supervision

T I O N S T O G E T H E R

Neva 1 Bridge, Russia (2002)
All cables equipped with VSL Friction Dampers

Koshiki Daimyojin Bridge, Japan (1993)
Technical consultation and supply of the prefabricated stay cables

Fred Hartmann Bridge, USA (1995)
Supply of stay cables and supervision at installation

Sunshine Skyway Bridge, USA (1986)
Supply of post-tensioning and stay cables. Cables anchored to the pylon by saddles and equipped with hydraulic dampers

Neva 2 Bridge, Russia (2007)
Supply and installation of stay cables with VSL Friction Dampers

Merida Arch Bridge, Spain (1991)
Supply and installation of the stay cables

TIONS TOGETHER

Zizkova Footbridge, Czech Republic (2007)
Installation of stays and post-tensioning

© Jose Cartellone Construct

Pastaza Bridge, Ecuador (2005)
Supply and installation of the stay cables

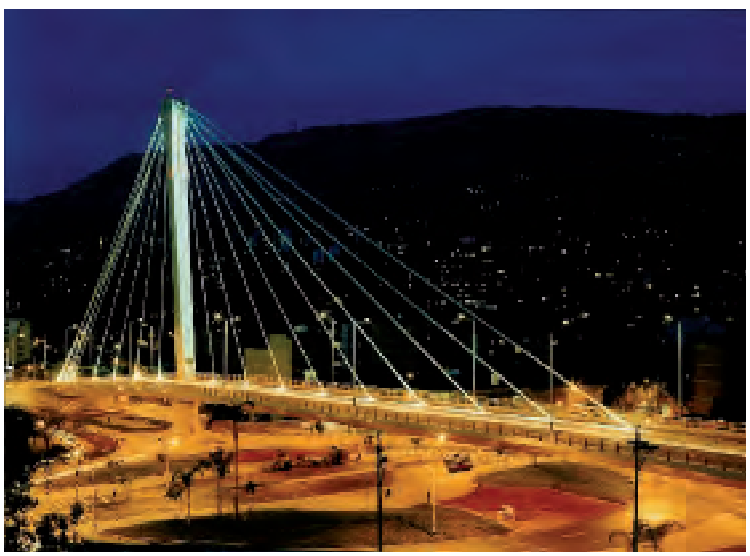
Rades-la-Goulette Bridge, Tunisia (2007)
Extensive equipment and services

Keppel Bay Bridge, Singapore (2006)
Supply and operation ofasting cells, erection of bridge deck, pylon construction, supply and installation of stay cables

Kien Hai Phong Bridge, Vietnam (2003)
Supply and installation of bearings and stay cables

Woonam Bridge, South Korea (2008)
VSL pylon saddles designed to allow cable replacement

Safti Bridge, Singapore (1995)
An inclined pylon is stabilised by back-stay cables



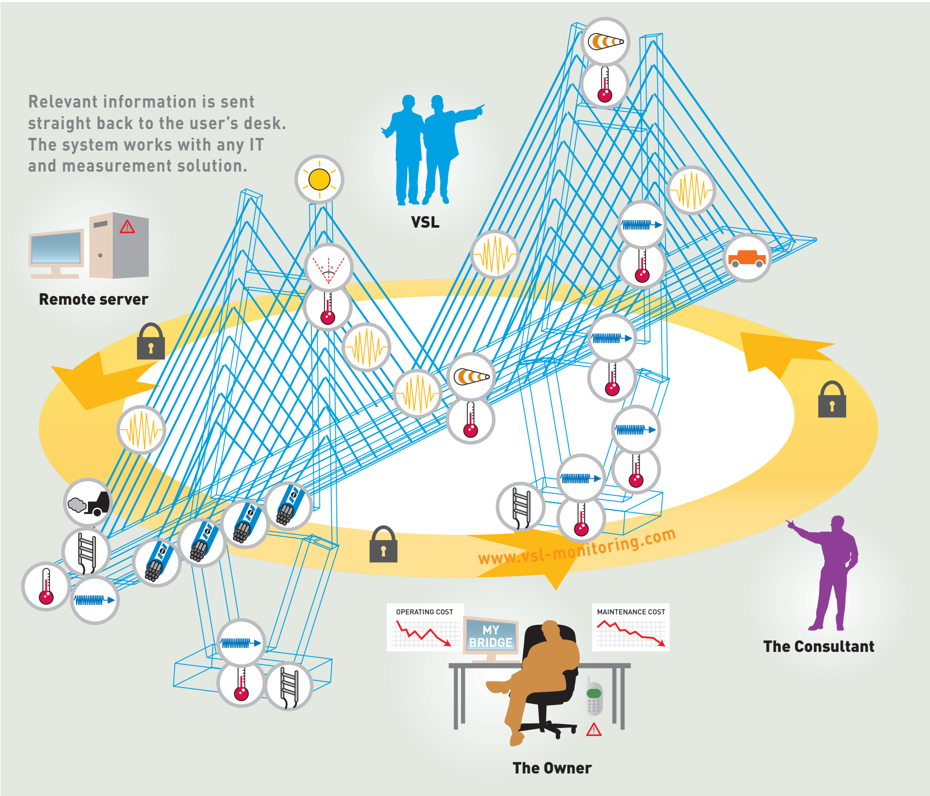
Peldar Bridge, Columbia (2002)
VSL Saddle and monitoring services

Liz Bridge, Portugal (2004)
18 stay cables installed in one week

T I O N S   T O G E T H E R

VSL SERVICES FOR STAY CABLE BRIDGE

MONITORING



Wind speed
measured in 3 directions
expressed in m/s

Temperature of cables, concrete
expressed in °C

Humidity
expressed in %

Rain intensity
expressed in mm

Sun intensity
expressed in W/m^2

Traffic
expressed in km/h, veh./h, t, type...

Air quality
expressed in CO, NO_x, O_3, SO_2...

Cable force
expressed in kN

Structural vibration
acceleration expressed in g
vibration modes expressed in Hz

Angular movement
two directions expressed in °

Stress and deformation
expressed in μm/m and in mm

Material ageing
expressed in Cl^-, mV...

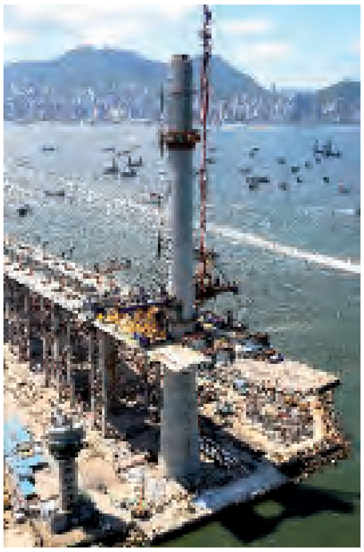
Inspection and investigation
paper and multimedia reports
laboratory or in situ tests

DeMon system allows wireless connection from client's office to any type of sensors on site through internet and wireless devices.

LAUNCHING GANTRIES

Shenzhen Western Corridor - Hong Kong

HEAVY LIFTING



Stonecutters Bridge - Hong Kong

DAMPERS

VSL dampers

GE CONSTRUCTION

CREATING SOLUTIONS TOGETHER

FORM TRAVELLERS & LIFTING FRAMES

West Tsing Yi - Hong Kong

REPAIR

Figueira Da Foz Bridge repair - Portugal

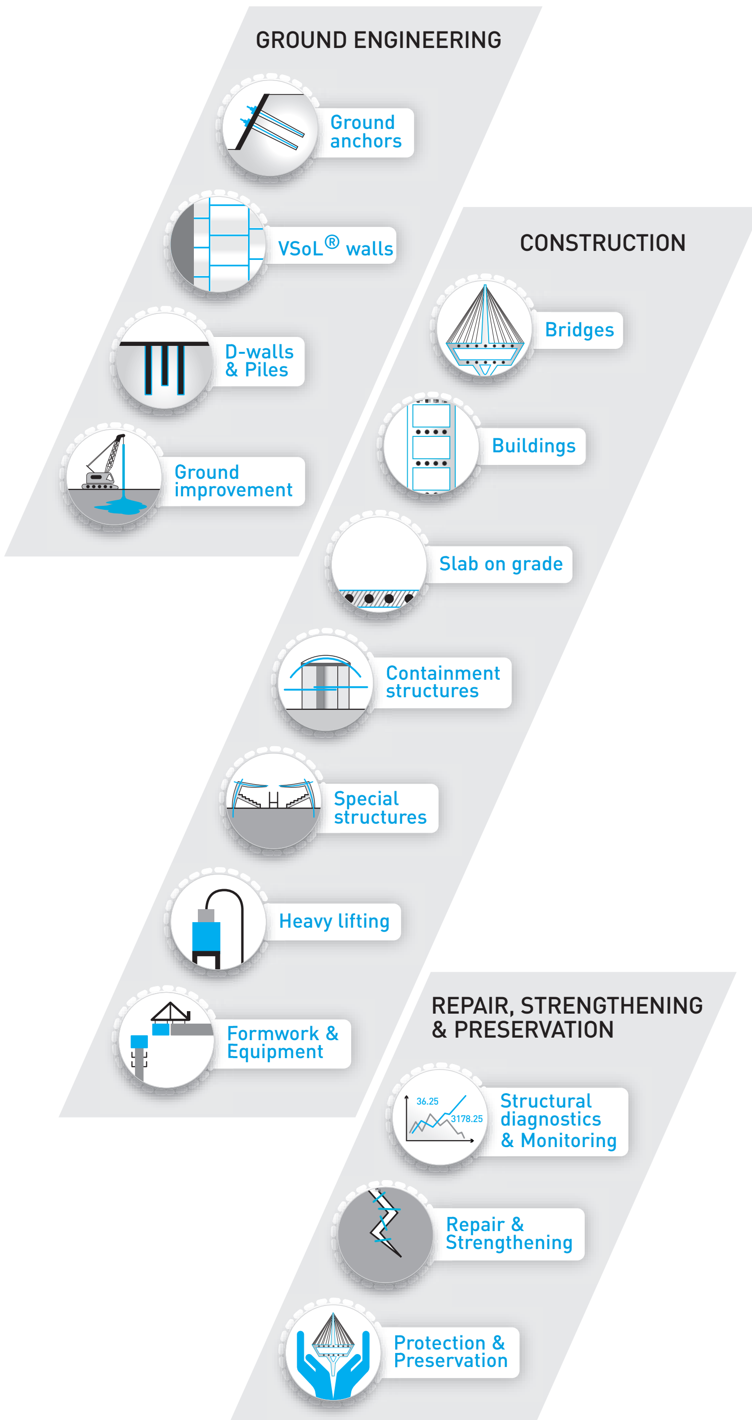
SADDLE

VSL SSI Saddle for a compact and slender pylon design

BEARINGS

Pot Bearing

Preloaded seismic bearing



SYSTEMS & TECHNOLOGIES

- Post-tensioning strand systems
- Bars & post-tensioning bar systems
- Stay cable systems
- Damping systems (stays & buildings)
- Ductal® UHP concrete
- Bearings & Joints

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REDLINE

spennteknikk

TOBE® FR4 pot bearing

CE
0402-CPD-496801

Designed according to EN 1337-5

BBR A Global Network of Experts
www.bbrnetwork.com

General

The new TOBE® pot bearing, the FR 4, is a further development of the recognized FR 3 type bearing. TOBE® has been the leading pot bearing on the Scandinavian market since the 1970s. The bearing solution is designed according to EN-1337-5 and it is CE-certified.

Application

TOBE® pot bearings are used as flexible bearing elements in road and railway bridges, offshore installations and other large building structures where major forces and movement must be absorbed.

Technical design

The TOBE® pot bearing is shaped as a cylinder/pot with a piston. Between the cylinder and the piston there is a temperature resistent rubber element that withstand very low temperatures. The rubber element is completely sealed-in and can be regarded a fluid medium under pressure, thus allowing rotational motion between the bearing parts.

TOBE® pot bearings are manufactured in three types, depending on function:

1. TOBE® F, fixed pot bearing
2. TOBE® A, multidirectional pot bearing
3. TOBE® E, unidirectional pot bearing

You may study a brief description of the varieties on the next page.

The benefits of TOBE® pot bearings

- Designed for resisting major vertical and horizontal loads
- Allow major rotation (tilt angle)
- Allow major movement
- Movable bearings (A and E) have very low friction coefficient
- Low structural height
- Can be replaced, dismantled or shimmed
- Can be safely inspected while in use
- Simple and safe assembly
- Designed in accordance with EN-1337-5, CE-certified
- Manufactured in Norway in accordance with NS-EN-9001
- Short delivery time
- Customized pot bearing solutions can be delivered at short notice

Quality control

KB Spennteknikk AS has developed an extensive quality control programme for the production of the TOBE® pot bearings. The company has been certified in accordance with NS—EN-9001 since 1994.

Load and movement ratings

The standardized bearing dimensions are designed for the following load limits:

Vertical, V_{max} from 520–33400 kN
Horizontal, H_{max} from 0–3260 kN

Please study the tables on pages 9 – 11 for a summary of the permitted load combinations, movements and dimensions for the various bearing types.

The standard TOBE® pot bearings are designed for a rotation/tilt angle not exceeding 20%. The unidirectional bearings (in the standard version) have a movement capacity of +/- 50 mm or +/- 100 mm. The multidirectional bearings are, in addition, designed for a transvers movement of +/- 20 mm.

TOBE® pot bearings can be designed and manufactured for considerably larger loads and movements upon request.

Surface treatment

All steel components in the TOBE® bearing will have a high quality surface treatment in accordance with the strictest requirements of both Norwegian and Swedish standards:

- Sandblasting to SA 2,5
- Metal spraying to 85 µm using Zn/Al = 85/15
- Sealing with epoxy sealer 15 – 25 µm
- Top coat with epoxy paint 100 µm. Top coat when mounting to steel structures, < 60 µm on contact surfaces

Other surface treatment specifications are available upon request.

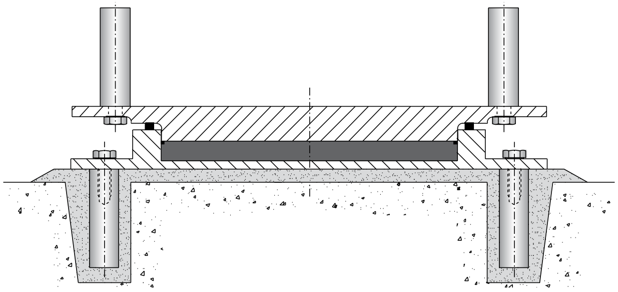
Replacements and adjustments

Any TOBE® bearing can be replaced without having to make any other structural or constructional changes. For the purpose of even more simplified replacement and adjusting tasks, the bearings can be equipped with additional bearing plates on top of and/or below the bearings.

Reading the movement indicator

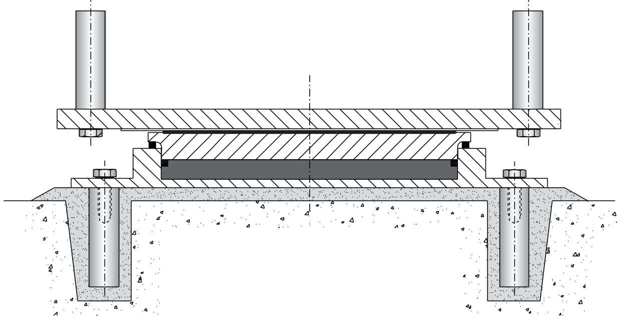
The movable TOBE® bearings (A and E) offer an indicator with a pointer that indicates the bearings´ accurate position at all times. The indicator can easily be read through binoculars, providing that it is placed visibly on the exterior of the structure. The indicator´s location should be decided at the ordering stage.

Fixed pot bearings, type F



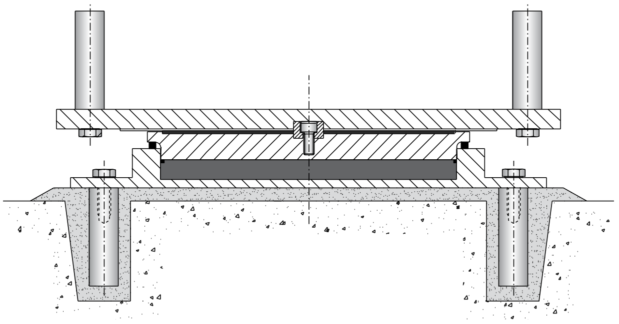
TOBE® fixed pot bearings are designed with two separate bearing plates secured with anchor bolts cast in to the upper and lower structure elements. For use in steel structures, the bearings are delivered without dowels and bolts. Between the bearing plates there is a rubber element which allows the bearing to rotate. The shape of the bearing will however ensure that the plates do not move horizontally.

Multidirectional pot bearings, type A



The multidirectional pot bearing option consists of three bearing plates. The upper and lower plates are fixed to the structures as for fixed bearings. Movement in the horizontal plane is achieved by the use of a sliding layer between the piston and the upper bearing plate. The sliding layer consists of a PTFE plate that is inserted into the top of the piston. A stainless steel plate is fixed to the upper bearing plate to ensure minimal movement friction.

Unidirectional pot bearings, type E



TOBE® unidirectional pot bearings are principally built in the same manner as the multidirectional option. However, the piston is fitted with a guiding wedge fitted in to the bearing´s upper plate, preventing transverse horizontal movement.

To the design engineer

TOBE® pot bearings are designed in accordance with EN-1337-5. The engineer must take into concideration the requirements for dimensioning of adjacent structures in accordance with the existing standards EN-1992-1-1:2004 and EN-1337-11.

1. Choice of bearing type

The following criteria must be known and taken into consideration:

1. The total vertical loads (ULS)
2. The total horizontal loads (ULS)
3. The amount of horizontal movement which the bearing must absorb
4. The rotation or tilt angle exposed to the bearing

While deciding what type of bearing to choose, the designing engineer must determine whether the bearings should be fixed or moveable in one or more horizontal directions. On page 6 there are suggested combinations of pot bearings for different kinds of bridge structures. It is also important to define whether adjacent structures are made of steel or concrete.

Always make sure to install the bearings horizontally or in a plane normally adjusted to the primary movement plane Thus the bearings will not be affected by rotation, and the rotation capacity will not be reduced by the primary load.

2. Determine bearing dimension

The pot bearing dimension is determined by the total vertical load, normally expressed as the total of dead load and live load, including load factors. Select the correct bearing dimension using tables on the pages 9 – 11. Make sure to check that the bearing of choice complies with all requirements in the horizontal maximum load calculations. The maximum horizontal load capacity depends on the minimum vertical load values filling the requirements listed in the tables.

IMPORTANT! The bearing table values are based upon the use of the concrete quality C40/50 according to the EN Standard.

Load combinations that exceed the tables´ standard values, required specialist design solutions. Please contact KB Spennteknikk AS.

3. Dimensioning of the foundation

To calculate the local foundation pressure, you need to know the pressure and the load influenced area. The following method is used:

a) Calculate the height of the grouting (Hu) according to EN-1337-11 pt 6.6

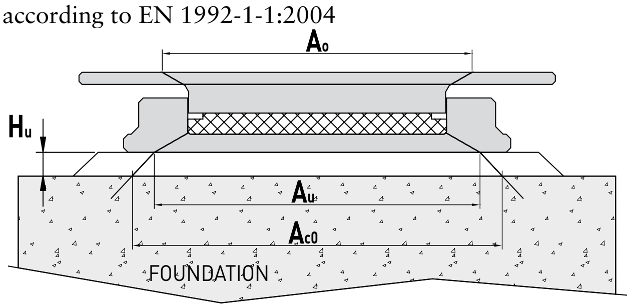
The thickness of the non-reinforced mortar bed (Hu) should not exceed 50 mm or:

$$H_U = 0{,}1 \ \frac{A_U}{2\pi \sqrt{A_U/\pi}} + 15 \qquad [mm]$$

Whichever is smaller. If the thickness of the mortar bed need to be increased, this should be consider by the designer. That may effect both reinforcement and dowels.

b) Calculate area against foundation (A_{C0})

according to EN 1992-1-1:2004



A_u and A_o are listed in the tables on page 9–11.

$$A_{c0} = \pi \left(\sqrt{A_U/\pi} + H_U\right)^2$$

c) Calculate local pressure in the foundation (FRDU)

according to EN 1992-1-1:2004 pt 6.7

$$F_{Rdu} = A_{c0} \cdot f_{cd} \cdot \sqrt{A_{c1}/A_{c0}} \leq \ 3{,}0 \cdot f_{cd} \cdot A_{c0}$$

In order to design the foundation, the engineer has to consider the total vertical and horizontal loads, as well as factors that occur as a consequence of pot bearing rotation.

4. Restraint moment

The restraint moment that influences the over- and underlaying structure is caused by:

1. Horizontal force
2. Angular change in the top part of the pot bearing
3. Friction between piston and pot (cylinder) wall

The restraint moment is found using this formula:

$$M_{TOT} = M_H + M_{emax} + M_{\mu max}$$

M_{emax} and $M_{\mu max}$ can be added up as the vectorial sum. However, a straightforward addition will always keep you safe.

M_H , M_{emax} and $M_{\mu max}$ are calculated as follows:

i) Calculating of M_H

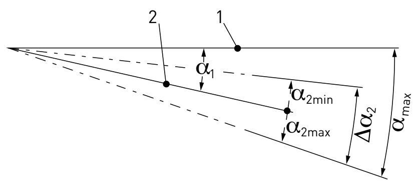
$$M_H = H \ x \ h \ [Nmm]$$

M_H = Moment due to horizontal force
H = Horizontal force in Newton (N)
h = The distance between the point of action of the horizontal force at the pot wall and the lower edge of the bearing in mm. Please see table on page 9. The h-values are the same for all 3 bearing types

ii) Calculating of M_{emax} (empirical formula)

$$M_{emax} = 32 \cdot D^3 \left(F_0 + (F_1 \cdot \alpha_1) + (F_2 \cdot \alpha_{2max})\right)$$

M_{emax} = Restraint moment (Nmm) from the rubber element
D = Rubber element diameter (please see table page 9)
α_1, α_{2max} – Rotation angle in rad. see figure below



1. Starting position after bearing installation
2. Position due to rotation α_1 caused by permanent loads

α_{2min} = Negative rotation caused by variable loads
α_{2max} = Positive rotation caused by variable loads
$\Delta\alpha_2$ = Range of rotation angle due to extreme positions of variable loads
α_{max} = $\alpha_1 + \alpha_{2max}$

$F_0 = 0{,}00263$, $F_1 = 0{,}3255$, $F_2 = 2{,}02$

F0, F1 and F2 are dimensionless coefficients determined by physical tests and valid only for TOBE® pot bearings.

iii) Calculation of $M_{\mu max}$

$$M_{\mu max} = 0{,}07 \ x \ H \ x \ D$$

$M_{\mu max}$ = Friction between the piston and the pot wall
H, D = Please see above

5. Horizontal loads diverging from table values

The horizontal force capacity is listed in the tables on pages 9–11. If the horizontal load exceeds the table values, the engineer should perform the following calculation:
If $V_{min\ actual} < V_{min\ table}$ the H_{max} must be reduced according to the following formula;

$$H_{max\ actual} = H_{max\ table} - 0{,}2\ (V_{min\ table} - V_{min\ actual})$$

H_{max} can however be increased provided that the bearing anchorage capacity is increased. Contact us for assistance if the loads diverse from the table values.

6. Range of bearing movement

The permitted range of movement for the movable pot bearing is determined based on the need for movement capabilities in the actual structure. Movements can be caused by factors such as temperature, shrinkage, creep, structural settings, backfilling and prestressing after installation.

The bearing should be designed to absorb and tolerate all kinds of movements that may be expected to influence the structure. Movement exceeding our standard range should be specified in the inquiry.

7. Movement friction

The inevitable force of friction is dependant on the specific surface strain on the PTFE plate. The frictional coefficient is reduced by increasing load and should be calculated in accordance with EN-1337-2:2004.

$$\mu_{max} = \frac{1{,}2 \cdot k}{10 + \sigma_{PTFE}} \qquad \text{where: } 0{,}03 < \mu_{max} < 0{,}08v$$

k = 1,0 for the Teflon disc [MPa]
σ_{PTFE} = The pressure on the Teflon disc [MPa]
Area Teflon disc (A_{PTFE}): See tables on pages 10 and 11 [mm^2].

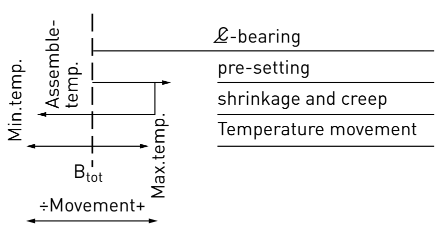
8. Pre-setting of pot bearings

When calculating the bearing specifications it is important to consider possible movements in the structure. The purpose of presetting is to make any likely future longitudinal movements symmetrical to the bearing´s centre line. By presetting the bearing, it is possible to offset temporary movements due to factors as described in section 6 above.

A pre-setting example:

e = K + BM – B_{TOT}/2
e = The pre-setting of the bearing
K = Calculated shrinkage and creep movements
BM = Temperature movement caused by changes in temperature after installation
B_{TOT} = Total temperature movement

Example of movement pattern:



9. Advice on pot bearing applications

While writing inquiry documents or direct orders for bearings, it will be appropriate to describe the technical requirements of the bearings before the complete dimensions and specifications. Thus, Spennteknikk can choose a type of bearing that will satisfy your price, operational and design requirements.

Please define these requirements prior to inquery:

a) Maximum vertical load with corresponding maximum horizontal load at ULS
b) Minimum vertical load with corresponding horizontal load at ULS
c) Maximum horizontal load with corresponding minimum vertical load at ULS
d) Rotation capacity
e) Movement capacity (movable bearings only)
f) Special requirements regarding geometry, adjacent structures, surface treatment etc.

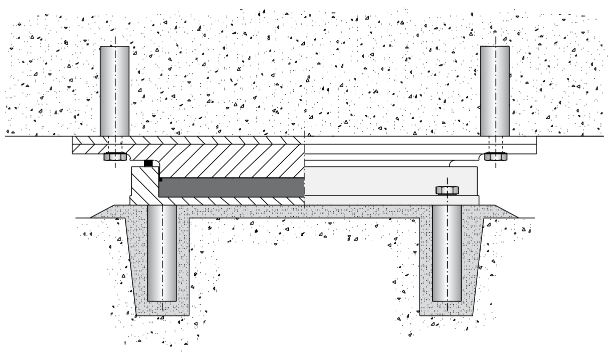
Additional bearing plates

All standard TOBE® pot bearings are replaceable. Under certain circumstances, e.g. possible future settings, it may be of great advantage to use additional bearing plates over and/or under the bearing to avoid problems. Thus the installation of adjustment plates and replacement of the pot bearing will be considerably simplified. The standard thickness dimension of the additional plate is 10 mm. Additional plates and adjustment plates can be delivered on request.

Simplified procedure for the installation of adjustment bearing plates:

1. Remove the bolts from the top bearing plate
2. Jack up the superstructure corresponding to the settings (+ 10–20 mm)
3. Place the adjustment plates
4. Carefully lower the superstructure
5. Replace the bolts and tighten them

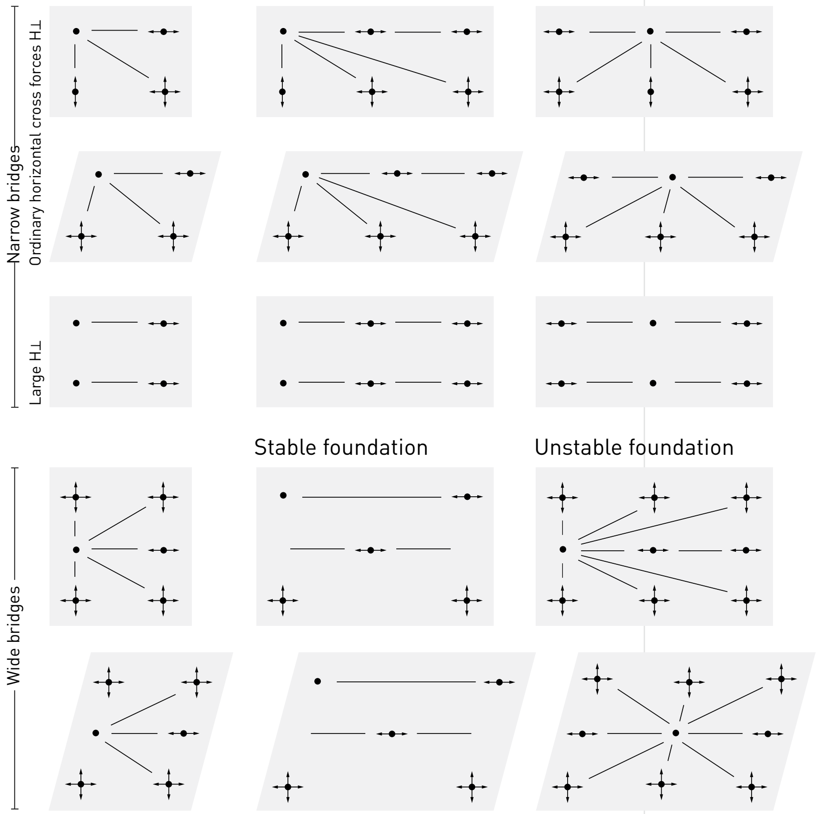
If settings exceed 20 mm, special adjustment plates should be used. Please contact us for assistance.



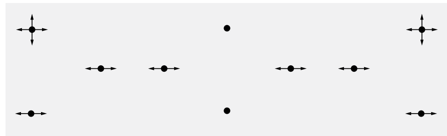
TOBE pot bearing applications

Legend: ● **TOBE F** Fixed pot — **TOBE A** Multidirectional pot bearing — **TOBE E** Unidirectional pot bearing

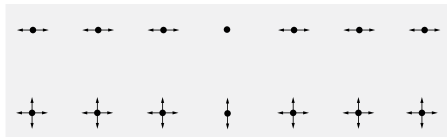
One span **Two spans**



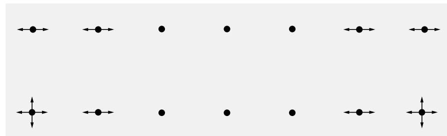
Several spans (straight bridge)
Extremely stable foundation



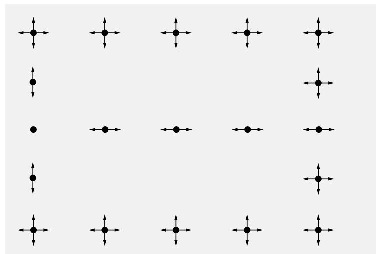
Stable foundation



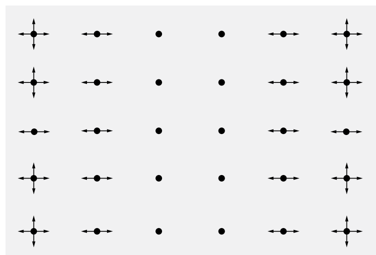
Unstable foundation



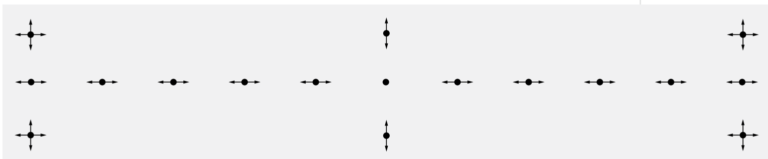
Several spans (straight bridge)
Stable foundation



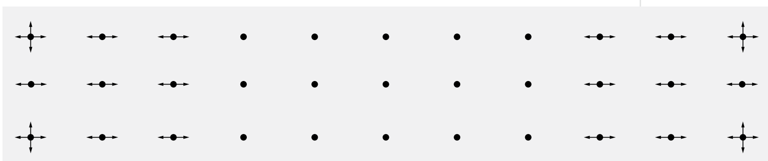
Unstable foundation



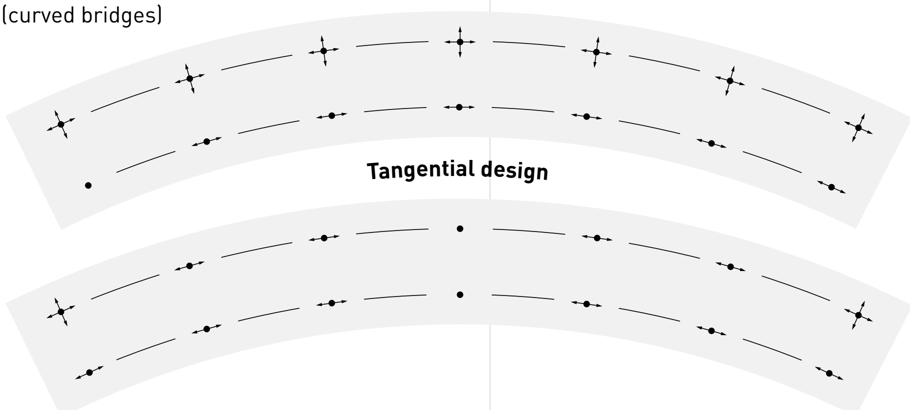
Stable foundation



Unstable foundation



Several spans (curved bridges)



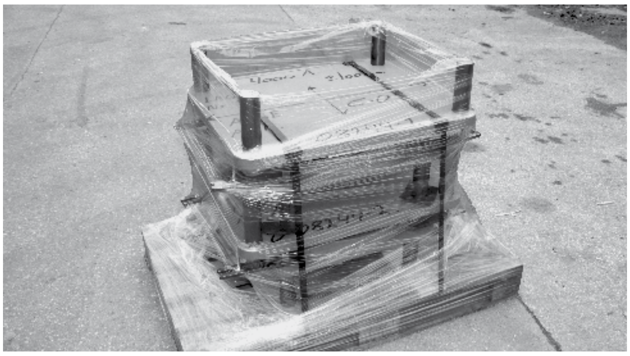
To the contractor/Client

When ordering pot bearings please be sure to provide the following information:
2. If required; desired presetting and direction
3. Desired marking on the surface of the bearing´s upper plate
4. Exact location of the pot bearing (axis, position etc)

If this informations are not available, the bearings will be supplied in a neutral position and with standard marking.

Delivery routines

The bearings are delivered solidly strapped to a shipping pallet, completely sealed in plastic. The receiver must have access to local transportation facilities for delivery to a temporary storage location. For a long term storage between delivery and installation, the bearings should be stored indoors.



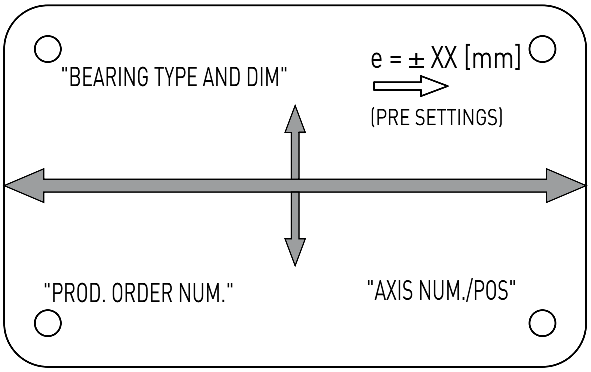
The marking of bearings

The pot bearings´ top plate is marked with the following information as standard:

Primary and secondary direction of movement

- Pre-setting information and arrow marking of direction (if pre-set)
- The bearing type and movement capacity
- The manufacturer´s order number
- Axis-number (option)
- The bearing´s position in the axis (option)

The figure below shows a typical marking of a multi-directional pot bearing TOBE® A.



Installation of bearings

An installation instruction guide accompanies every delivery.

Installation information

The TOBE® bearings should be installed in accordance with the designer's instructions shown on the construction drawings.

Drawings should indicate:

- Recesses for anchor bolts, with dimensions
- Drawn-in top plate, with dimensions and placement direction for the bearing.
- Cross-section of the pot bearing indicating presetting values and direction
- Contour height of top bearing
- Requirements for grouting materials and execution
- All other possible information of importance for the bearing installation

General description

The TOBE® pot bearings are supplied with four pcs of 150 mm long, Ø35/50 mm dowels both on the upper and the lower bearing plates. For use in steel structures, the bearings are delivered without dowels and bolts. The dowel position is described in the tables on pages 9 – 11, referring to the different bearing types.

Bearing direction during installation

Fixed bearings can be oriented in any direction, unless the engineer has described an imperative placement. Uni- (E) and multidirectional (A) pot bearings have sliding plates that should be oriented parallel to the bearing´s movement direction. The movement directions are marked on the upper side of the bearing with a primary direction indicated by an arrow in the lengthwise direction of the sliding plates (A and E), and a secondary direction marked with a short arrow crosswise (type A only). The movement directions are defined by the engineer, and may occur both lengthwise and crosswise.

To provide safe transport and installation, the bearings are temporarily fastened with four pcs of special transport fittings, one in each corner of the bearing (type A and E). Once the bearing installation is completed, the fittings are removed and the bolts are tightened. The fixed bearing (type F) is temporarily fastened with six pcs of M6 aluminium bolts between the pot and the piston. These bolts can either be cut off or left in place, as they will break upon the very first rotational movement. The presetting is marked on the bearings. Carefully check that the presetting is correctly positioned in accordance with the designer's instructions.

Recesses

When concreting the bearing´s foundation, form work recesses for the dowels, either using Ø100 mm, depth 150–200 mm, or squares measuring 150 x 150 mm, are needed.

Installation
Before casting the bearing in concrete, it should be carefully adjusted to its correct position at four points, using wedges. The bearing should be level, stable and in correct height. To establish grouting dimentions, please read the instructions on page 4, item 3a. If in any doubt, contact the manufacturer.

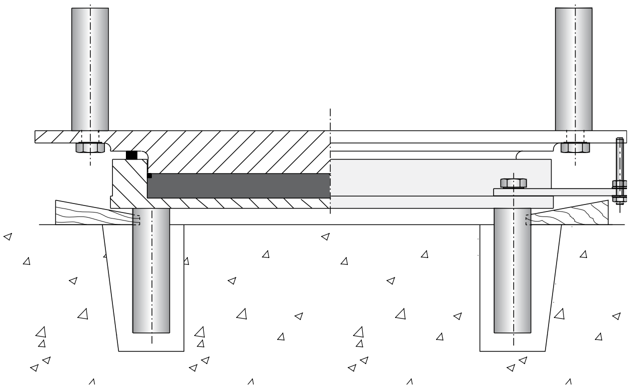


Figure: Adjusted bearing using wedges

Grouting
Use high-grade low shrinkage mortar for grouting. Be sure to follow the manufacturer´s instructions for use. In frosty conditions, choose a frostproof option.

Use the following procedure for grouting beneath the pot bearing:
1. Fill the recesses for the mounting fittings almost to the top
2. Remove the wedges after the mortar has set
3. Build form work with extra height around the bearing. For bearings smaller than TOBE FR 50 the mortar may be packed underneath the bearing plate and tamped thoroughly or mortar may bee injected.
4. Pour liquid mortar from one side into the formwork in order to squeeze out all air
5. For bearings in sizes from TOBE® FR50 and larger, the mortar should be injected to obtain a good result.

Study EN-1337-11 for a complete overview of the grouting requirements.

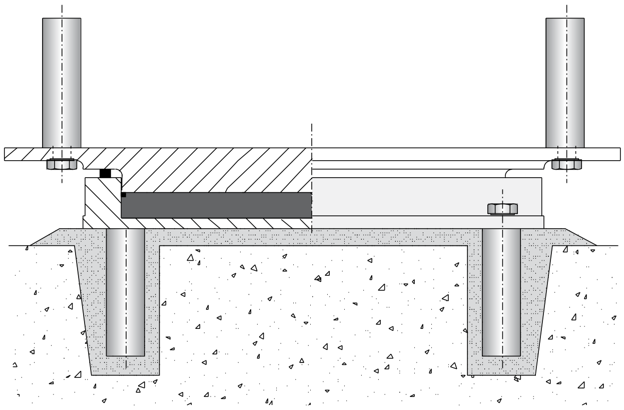


Figure: Bearing after grouting

Presetting the slide bearing
As a rule, presetting of the slide bearings should be performed by the manufacturer. Presetting may however be carried out on the construction site by loosening the mounting bolts and sliding the top plate of the bearing into the right position. IMPORTANT! Avoid lifting the bearing plate while sliding as this may cause the Teflon plate to move.

The location of the movement indicator and the pointer should be decided at an early stage, thus making the reading of the indicator best possible. This decision should be taken in the ordering process.

IMPORTANT! The presetting location must be thoroughly checked prior to the final installation works.

The superstructure
While pouring the superstructure, the top plate of the bearing will function as a part of the formwork. The space between the pot walls and the formwork must be properly sealed and protected to avoid contaminating the pot bearing with concrete during pouring. Pour the concrete directly against the bearing when pouring the rest of the superstructure.

After completion
When the superstructure has set and sufficiently hardened, remove the formwork and the transportation devices. Tighten all bolts. Clean up dirt and remaining of concrete.

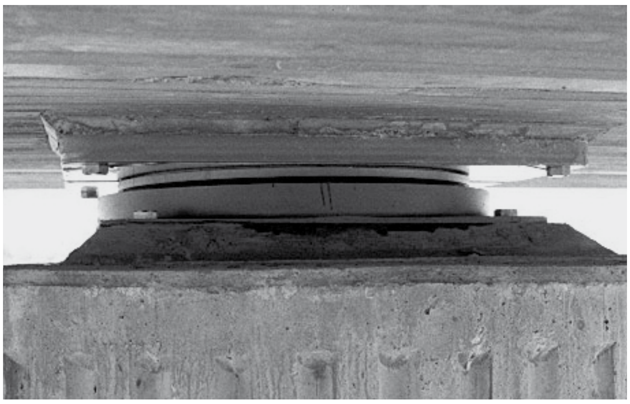
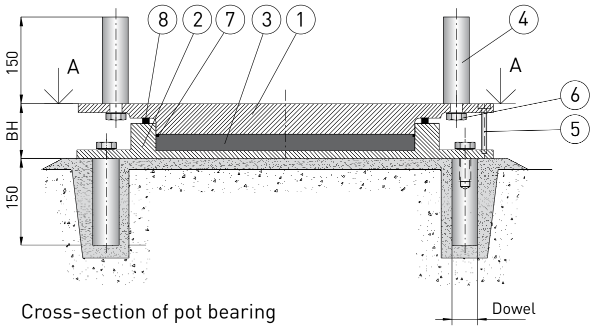


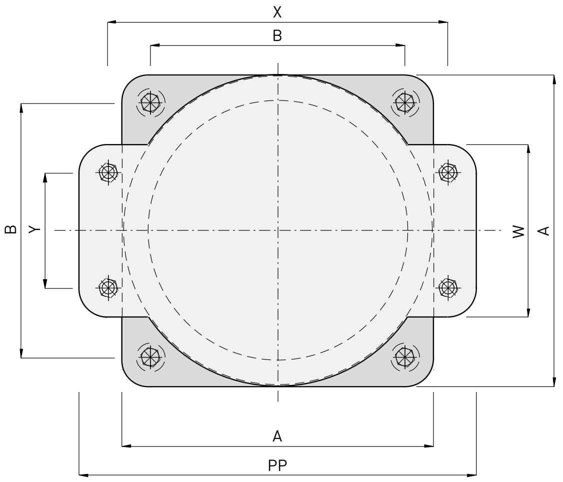
Image: Finished bearing

Jacking up the structure
While planning the project it is advisable to consider any possible future needs to jack-up the structure. You should plan for enough storage room for heavy-duty jacks near the bearing points. The lifting points for the jacks must be parallel and the jack placement foundation must be dimensioned to endure the high pressure from the jack sockets.

Spennteknikk offers repair and bearing replacement services, performed by our own professionals.



Cross-section of pot bearing



A – A

Fixed pot bearing Type F

1. Piston
2. Pot
3. Rubber element
4. Anchor bolt (dowel)
5. Transport devices
6. Bolt
7. Sealing ring
8. Rubber sealing

Forces:

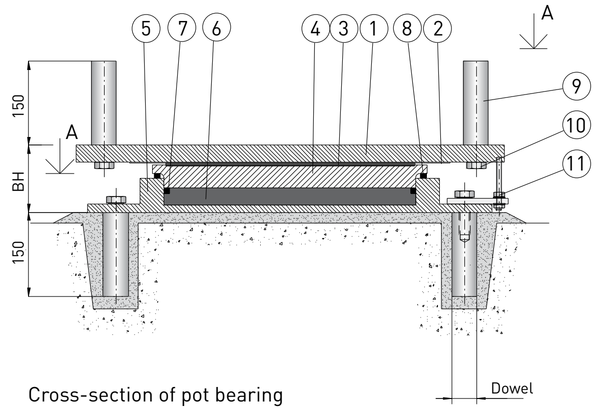
V_{max} = Maximum allowable vertical load (ULS), with concrete C40/50 acc. CEN (B40 acc. NS)
H_{max} = Maximum horizontal load (ULS)
V_{min} = Necessary minimum vertical load with Hmax kN

Maximum allowable tilt angle (rotation) = ± 20‰

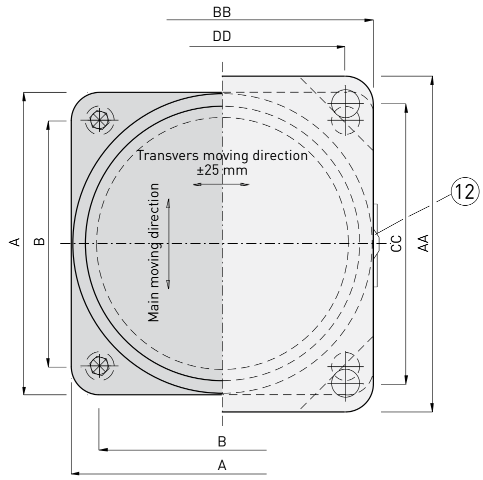
TOBE® pot bearings can be designed and manufactured for considerably larger loads and movements upon request.

Load table for fixed pot bearings at ULS

Type		10	20	30	40	50	60	70	80	90	100	110	120	130
V_{max}	kN	520	1590	3040	4690	6090	7670	9060	10570	13480	16250	21450	27400	33400
V_{min}	kN	60	380	1210	2070	2790	3400	4070	4670	5670	6830	8110	9830	11360
H_{max}	kN	220	420	720	950	1130	1280	1450	1600	1850	2140	2460	2890	3260
Dimensions fixed bearing (all measurements in mm)														
A	mm	200	295	390	470	535	590	640	685	770	840	955	1070	1170
B	mm	149	232	315	386	440	470	510	540	600	650	730	810	880
W	mm	115	165	200	250	300	300	350	375	425	450	500	550	600
X	mm	230	335	440	520	595	655	700	745	830	910	1015	1135	1230
Y	mm	64	102	125	166	205	205	255	280	330	355	405	455	505
BH	mm	56	56	67	72	81	90	93	101	108	118	130	145	161
PP	mm	281	398	515	604	690	750	795	840	925	1005	1110	1230	1325
Ø	mm	35	50	50	50	50	50	50	50	50	50	50	50	50
Bolt dim.		M16	M20	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24
h	mm	29,5	29,9	36,2	42,2	47,6	52,0	55,5	58,8	66,0	71,2	79,2	88,8	97,0
Rubber D	mm	120	210	290	360	410	460	500	540	610	670	770	870	960
Ao	mm^2	17671	45239	80425	119459	152053	188574	220618	255176	321699	384845	502655	636173	769769
Au	mm^2	20503	49706	86345	126652	162621	200323	236287	275218	347768	417267	544134	692919	837679
Weight	kg	20	43	70	96	129	164	193	236	301	384	529	722	955



Cross-section of pot bearing



A – A

Multidirectional pot bearing Type A

Movement	**Primary**	**Secondary**
	± 50 mm	±20 mm
	±100 mm	±20 mm

1. Sliding plate
2. Stainless steel plate
3. Teflon plate
4. Piston
5. Pot
6. Rubber element
7. Sealing ring
8. Rubber sealing
9. Anchor bolt (dowel)
10. Bolt
11. Transport devices
12. Indicator/millimetre scale

Forces:

V_{max} = Maximum allowable vertical load (ULS), with concrete C45/55 acc. CEN (B45 acc. NS)
H_{max} = Maximum horizontal load (ULS)
V_{min} = Necessary minimum vertical load with Hmax

Maximum allowable tilt angle (rotation) = ± 20‰

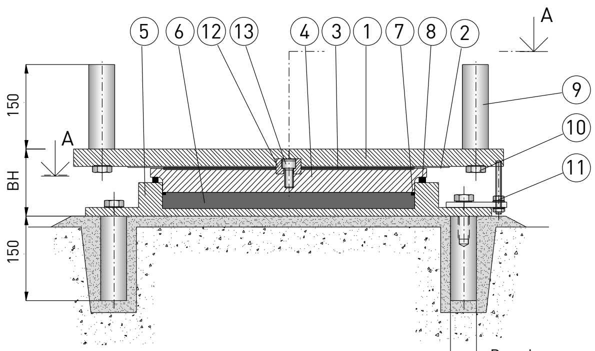
TOBE® pot bearings can be designed and manufactured for considerably larger loads and movements upon request.

Load table for multidirectional pot bearings at ULS

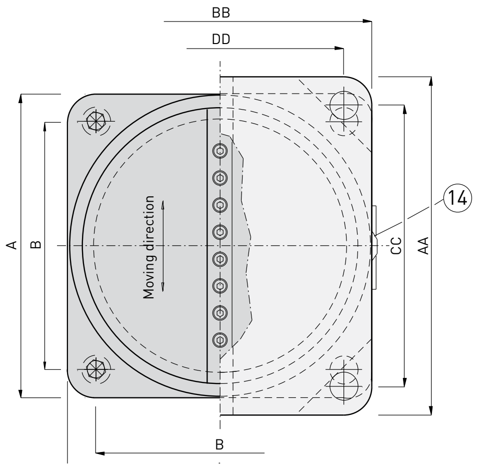
Type		10	20	30	40	50	60	70	80	90	100	110	120	130
Vmax	kN	520	1590	3040	4690	6090	7670	9060	10570	13480	16250	21450	27400	33400

Dimension table for multidirectional pot bearings (all measurements in mm)

A	mm	200	295	390	470	535	590	640	685	770	840	955	1070	1170
B	mm	149	232	315	386	440	470	510	540	600	650	730	810	880
AA ± 50	mm	300	420	495	560	635	675	710	745	805	855	945	1025	1105
CC ± 50	mm	249	357	420	476	540	580	615	650	710	760	850	930	1010
AA ± 100	mm	400	520	595	660	735	775	810	845	905	955	1045	1125	1205
CC ± 100	mm	349	457	520	576	640	680	715	750	810	860	950	1030	1110
BB	mm	200	300	380	450	500	550	590	630	700	760	860	960	1050
DD	mm	149	237	305	366	405	455	495	535	605	665	765	865	955
BH	mm	72	78	88	97	102	111	114	118	129	139	151	165	177
Ø	mm	35	50	50	50	50	50	50	50	50	50	50	50	50
Bolt dim.	mm	M16	M20	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24
A_{PTFE}	mm^2	14314	33979	57256	98980	132025	173494	204282	237583	301907	363168	477836	608212	738981
Ao	mm^2	27964	59634	89488	140232	179113	226949	261981	299526	371278	438905	564184	705171	845490
Au	mm^2	20503	49706	86345	126652	162621	200323	236287	275218	347768	417267	544134	692919	837679
Weight ± 50	kg	25	60	94	134	170	213	248	287	376	468	636	847	1103
Weight ± 100	kg	28	65	100	141	178	221	257	297	387	480	649	862	1119



Cross-section of pot bearing



Unidirectional pot bearing Type E

Movement

± 50 mm
± 100 mm
No transversal movement

1. Sliding plate
2. Stainless steel plate
3. Teflon plate
4. Piston
5. Pot
6. Rubber element
7. Sealing ring
8. Rubber sealing
9. Anchor bolt (dowel)
10. Bolt
11. Transport devices
12. Wedge (guide)
13. Bolt
14. Indicator/millimetre scale

Forces

V_{max} = Maximum allowable vertical load (ULS), with concrete C40/50 acc. CEN (B40 acc. NS)
H_{max} = Maximum horizontal load (ULS)
V_{min} = Necessary minimum vertical load with H_{max} kN
Maximum allowable tilt angle (rotation) = ± 20‰

TOBE® pot bearings can be designed and manufactured for considerably larger loads and movements upon request.

Load table for unidirectional pot bearings at ULS

Type		10	20	30	40	50	60	70	80	90	100	110	120	130
V_{max}	kN	520	1590	3040	4690	6090	7670	9060	10570	13480	16250	21450	27400	33400
V_{min}	kN	80	380	1170	1910	2630	3130	3530	3950	4650	5250	6250	7010	8000
H_{max}	kN	160	420	720	910	1090	1210	1310	1410	1590	1740	1990	2180	2400

Dimension table for unidirectional pot bearings (all measurements in mm)

A	mm	200	295	390	470	535	590	640	685	770	840	955	1070	1170
B	mm	149	232	315	386	440	470	510	540	600	650	730	810	880
AA ± 50	mm	300	420	495	560	635	675	710	745	805	855	945	1025	1105
CC ± 50	mm	249	357	420	476	540	580	615	650	710	760	850	930	1010
AA ± 100	mm	400	520	595	660	735	775	810	845	905	955	1045	1125	1205
CC ± 100	mm	349	457	520	576	640	680	715	750	810	860	950	1030	1110
BB	mm	160	260	340	410	460	510	550	590	660	720	820	920	1010
DD	mm	109	197	265	326	365	415	455	495	565	625	725	825	915
BH	mm	84	99	98	108	113	121	124	132	139	149	161	175	187
Ø	mm	35	50	50	50	50	50	50	50	50	50	50	50	50
Bolt dim.	mm	M16	M20	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24	M24
A_{PTFE}	mm^2	9859	25451	46168	84425	115215	154224	183372	215033	276487	335288	445856	572132	699211
Ao	mm^2	27964	61143	98908	151964	192342	241810	277930	316563	390220	459480	587479	731187	873954
Au	mm^2	20503	49706	86345	126652	162621	200323	236287	275218	347768	417267	544134	692919	837679
Weight ± 50	kg	25	66	99	145	185	229	268	318	405	501	680	903	1139
Weight ± 100	kg	27	70	106	153	194	239	278	329	417	515	696	921	1159

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MAURER Swivel-Joist Expansion Joint

The MAURER Swivel-Joist Expansion Joint is an enhancement of the Girder Grid Expansion Joint, considerably adding to the range of application of the MAURER Modular Expansion Joints. When large and complex movements are required then for geometrical and economic reasons the use of Swivel-Joist Joints are to be preferred rather than Girder Grid Joints.

Also in the case of restricted space, for instance in steel bridges and with the replacement of old rolling leaf type joints, the application of the watertight Swivel Joint is advisable.

The MAURER Swivel-Joist Expansion Joint's versatile movability makes it suitable for variable deformations along the bridge structure . The joint cannot only follow the main movement of the bridge in carriageway direction but also distinctive movements in the 2 spatial directions perpendicular to the main direction. Even rotations of the bridge about the three spatial axes are easily coped with.

The edge beams run parallel to the structural edges. In order to avoid material fatigue, the traffic loads are transmitted to the adjoining reinforced concrete structure via anchor plates which are rigidly connected to the edge beams.

Dependent on the size of movement numerous centre beams are arranged between the edge beams.
The center beams slide on obliquely arranged swivelling support bars, resting on elastic sliding bearings. Lift-off from the sliding bearing is prevented by means of a prestressed sliding spring that is arranged in the support stirrup underneath. Only in the joist-box (i.e. at the edge), the sliding spring is placed above the support bar. Stirrups provide constant prestressing that is geometrically controlled.

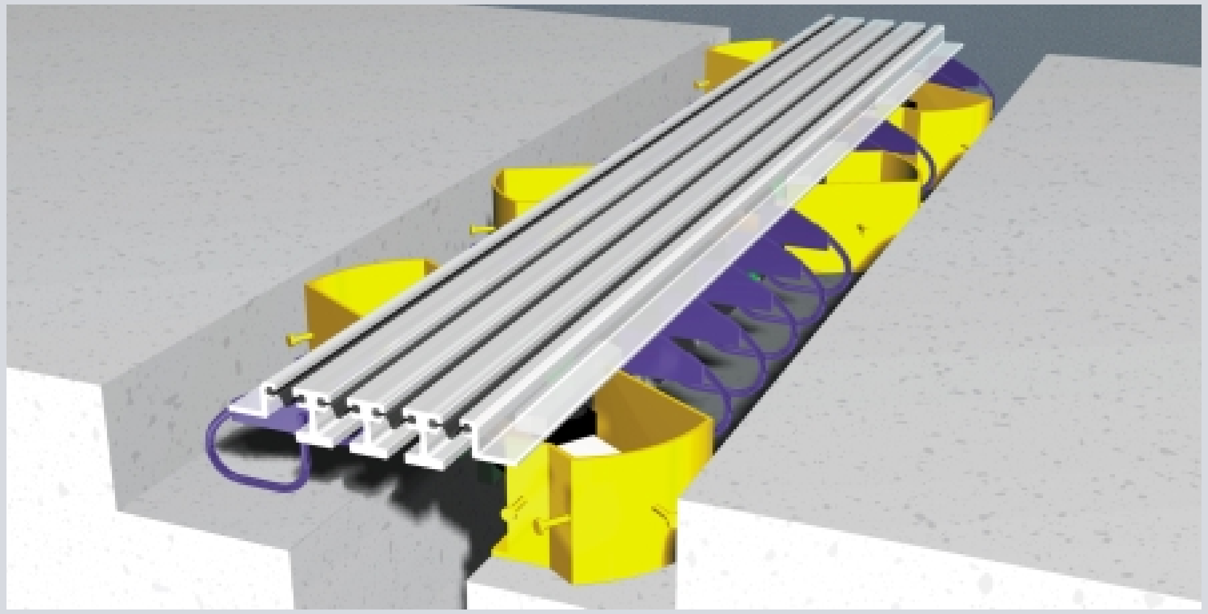
Vehicles travelling over the expansion joint transmit vertical and horizontal loads to the centre beams. The section forces resulting from the eccentric wheel loads are transmitted to the support bars (via prestressed sliding bearings) by means of the centre beams that act as continuous girders with torsionally elastic support. From there the forces are transmitted into the edges of the structure.

The bulbous-shaped EPDM strip seal is installed in a claw in the edge beam and centre beams without the need for additional clamping bars. The connection is watertight, with the sealing element set below the road surface level. This way it is protected against direct wheel or snowplough contact.
As a rule, the admissible horizontal displacement of the strip seal in carriageway direction is 80 mm.
With its preformed articulated section it is possible to move the strip seal in direction of the carriageway without any appreciable strain.

Replacement of a Roller Leaf Joint by a MAURER Swivel Joist Expansion Joint

Installation of the expansion joints is carried out in total length (i.e. in 1 piece) into the prepared recess. The structural connection shall be made in accordance with the rules of reinforced concrete construction and/or steel construction. The installation is completed with the connection of the waterproofing, followed with asphalting.

Type DS 320 displacement of the support bar on both sides



d.) Opening movements

The admissible gap width, which as a rule is 80 mm, can be exceeded during seismic action. The control elements, following the "theorem on intersecting lines", enable every opening condition of the expansion joint. By adapting the length of the support bars, opening conditions of whatever magnitude can be accommodated without strain. The sealing element will be adapted such as to follow the combined earthquake movements without the risk of unfolding. If for economic reasons the working range of the sealing element shall be limited, then by simple means this original limit can be restored again after the quake is over.

e.) Closing movements

When the expansion joint or the structural gap closes, there might result damages or even breakdown of the structure. For better protection of the bridge structure, MAURER SÖHNE has developped a so-called "fuse box" in addition to the new-style Seismic Expansion Joint. If the expansion joint should close in case of a quake, predetermined breaking points will be activated. The anchorage system disengages alongside a ramp according to a defined failure load and will return to its original position as soon as the quake is over. Stoppers provide temporary fixation of the position. Emergency vehicles can pass the joints. However, the anchorage will have to be reconstructed. An application of a fuse box can - as the case may be considerably reduce the number of sealing elements required.

University of Berkeley/ California Testing equipment

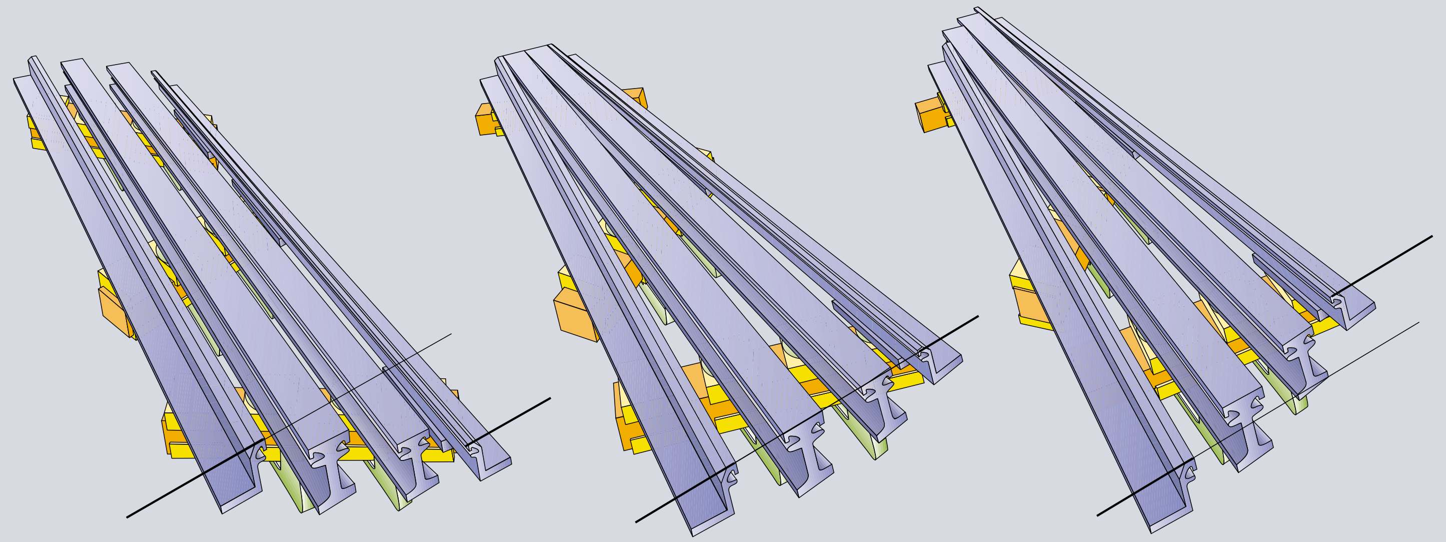
maximum transverse displacement

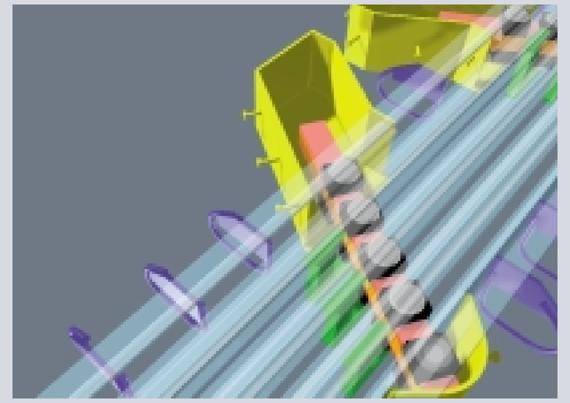
f.) Proof by testing

The behaviour of the MAURER Seismic Expansion Joint was tested at the University of Berkeley / California, actually the only institution capable to do such tests.
A test specimen of type DS 560 in scale 1:1 was subject to displacements of extremely high velocity and changing directions, at the same time simulating a variety of recorded seismic patterns.

Simultaneously longitudinal and transverse displacements of 1120 mm, coupled with a vertical offset of up to 6 %, were applied at resulting velocities of up to approx. 1600 mm/s. Even after imposing 30 full seismic patterns, no damages could be detected.

Control of Swivel-Joist Expansion Joints

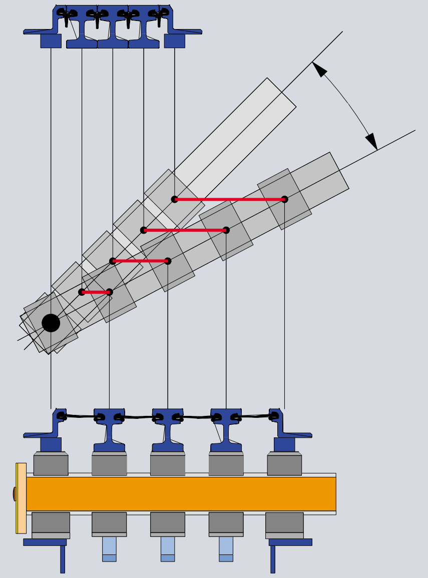




Type DS320
Control of the movement of
support bars to one side

On the one hand, rigid control mechanisms guarantee an exact allocation of the total movement to the individual gaps, and this mechanism also employs a clearly defined support system. But on the other hand such a rigid control is prone to strains that are caused by unplanned and unexpected movements, such as dimensional tolerances, difference in temperature in the respective members of the joint, and deviations from the designed movement. Any support system that neither accepts dimensional tolerance nor is prestressed resiliently, gives cause to strong noise emission and high wear. For this reason, modern modular joints employ a resilient control system. Usually this is achieved by plastic springs that are either being deformed along their longitudinal axis or by means of shear deflection. The individual center beams are connected by such springs. Thus we have several chains of sequentially arranged springs. As it is the case with such a system, the total resulting stiffness is a function of the particular number of center beams, or modules that are connected by this way.

One exception is the swivel joint system that is being controlled by guided and shear-resilient torsion hinges. This system has all the advantages of the exact scissor control system, but, due to its shear resilience, in addition the swivel joint system can also compensate dimensional tolerances and strains. Because each center beam is controlled individually, the stiffness of the horizontal support system is independent of the number of modules, or center beams. A swivel joist system employs a control mechanism with parallel arranged springs.



If the superstructure moves, the support bars will be pushed through the swivelling guiding bearings and thus experience a swivel movement. Due to the fixed distances of the torsion elements, this swivel movement gives rise to an almost even allocation of the total movement to the individual gap openings.

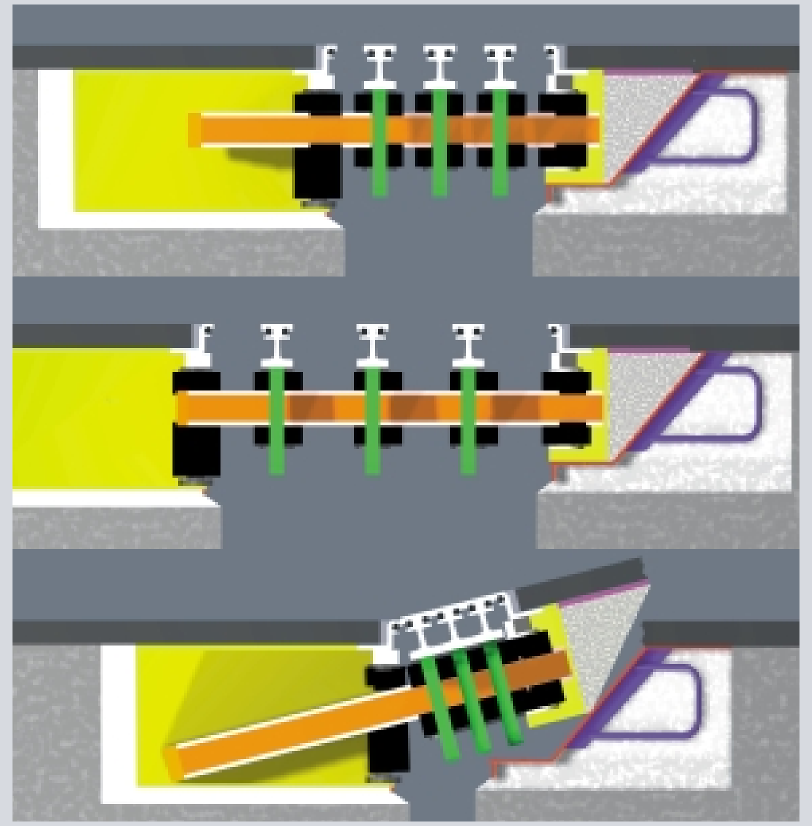
For large and irregular movements (e.g. from earthquake) there is no alternative to the MAURER Swivel-Joist Expansion Joint.

MAURER Seismic Expansion Joint

DS320
with fuse box,
Maximum
opened operating position
(s=80 mm)

Maximum opened
seismic position
(e.g.: s=150 mm)

Fuse box released
by earthquake,
when gaps closed



a.) General

There is a demand for reliable and economic solutions to cope with seismic strains. For operating condition the MAURER-SeismicExpansion Joint is dimensioned like a Swivel Joist Expansion Joint, geometrically adapted to the seismic movements. By this, the number of sealing elements as well as the wearing parts and finally the price are minimized. All movements are transmitted without constraints or damages.

b.) Direction of movement

The direction of movement is only restricted by geometrical obstacles in the support box. The unique Swivel Joist design allows for all kind of adaptations.

c.) Acceleration

Conventional modular expansion joints are controlled by springs that are arranged in series. Due to the mass inertia of the centre beams, seismic accelerations bring about inadmissible gap width deviations which finally destroy the supporting structure. In case that gap width delimiters should be provided here, the admissible opening of the Expansion joint is then however restricted to operating conditions only. The centre beams of MAURER-Seismic Expansion Joints are arranged in parallel, which means that each centre beam moves independently and hence it follows that there only minor additional gap width deviations will occur.

Earthquakes can generate structural movements which are considerably larger, many times quicker and much more complex in their direction than those under normal operational conditions. That is why applications of that kind require particular adaptation of the expansion joint.

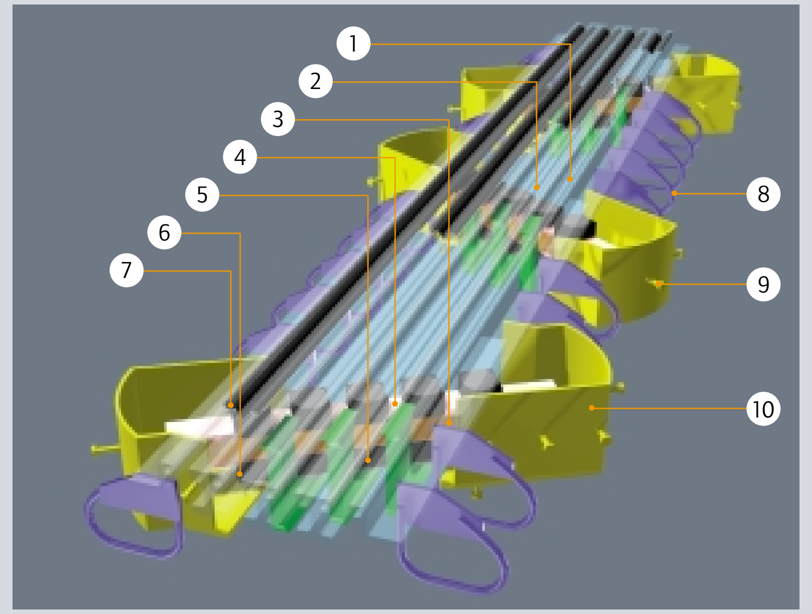
The conventional requirements set to the operating condition are irrelevant during seismic action. Of particular importance is, however:

- maintaining the serviceability of the structure after the earth quake at least for emergency vehicles as well as
- protection of the structure from impact damages caused by closing movements during the earthquake.

As a rule, conventional expansion joint systems can not fulfil these requirements. They are designed for movement sizes and directions under service condition. Whereas surpassing the admissible single gap widths during the quake is not dangerous in itself, this will cause the control system to be destroyed, as well as the mechanical gap width delimiters and the supporting elements. During seismic action the horizontally and/ or vertically undefinable direction of movement will eventually result in a blockage and destruction of the expansion joint. Due to high accelerations during a quake the sliding support elements are destroyed. The result will be a service breakdown of the bridge which is of vital importance for all emergency vehicles.

Employing a long and superior performance history in normal service conditions, the Swivel-Joist Expansion Joint had been further enhanced such as to also fulfil the aforementioned seismic requirements.

Design Principles and Main Components



Technical approval and independent
periodical inspection acc. to TL/TP-FÜ

Continuous in-house and field quality control, the use of high-grade materials, a quality assurance system in complying to DIN EN ISO 9001 as well as an environmental management system according to DIN EN ISO 14001 ensure the high standard of MAURER Swivel-Joist Expansion Joints.

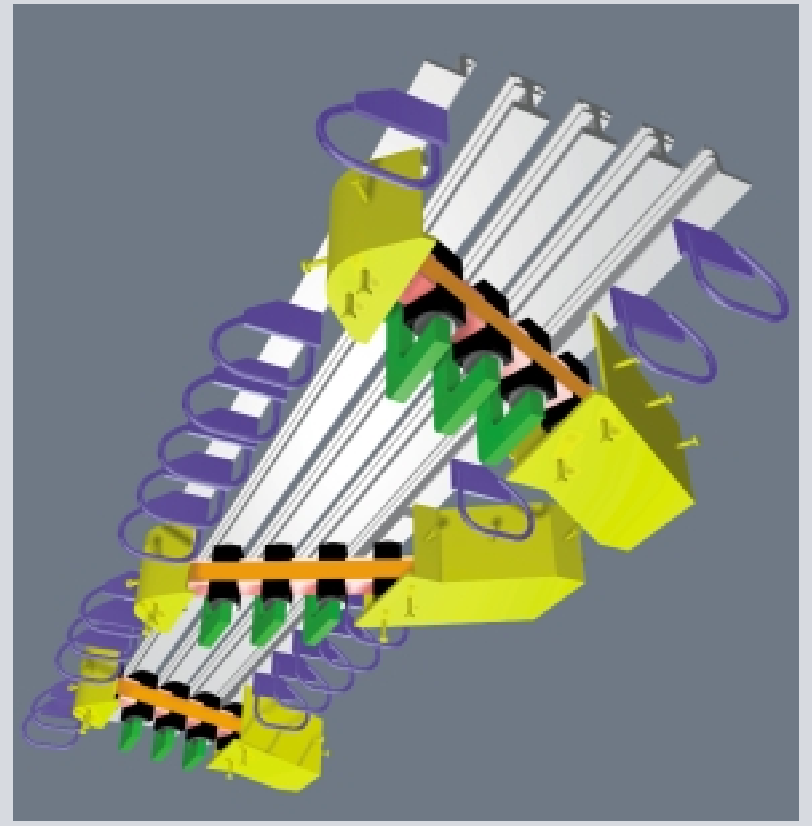
All design elements of MAURER Expansion Joints are engineered in high-quality materials. All synthetics employed feature excellent resistance to ageing, wear, and show a superior performance to all kinds of environmental impacts. Relaxation of the bearing elements is insignificant even after decades of service. The sealing elements are insensitive to physical stress.

National regulations are to be taken into account in the choice of the corrosion protection system. We recommend using two-coat zinc-rich paint as the primer and epoxy-based micaceous iron ore as the finishing layer.

Designation	Description
Supporting Elements	
1 edge beam	hot-rolled steel grade S 235 JR G2 with precision tolerances combining good weldability with notch toughness. Can be both shop and site butt-welded.
2 centre beam	hot-rolled steel grade S 355 J2 G3 with precision tolerances combining good weldability with notch toughness. Can be both shop and site butt-welded by a patented system.
3 support bar	steel grade S 355 J2 G3, machined for precision tolerances.
Supports	
4 sliding plate	stainless steel in bridge bearing quality material-no. 1.4401, sliding surfaces ground and polished.
5 sliding spring	natural rubber with vulcanized steel plates. Sliding surfaces of high strength PTFE sliding material.
6 sliding bearing	chloroprene-rubber reinforced with vulcanized steel plates, according to Bridge Bearing Standard DIN 4141, part 14. Sliding surfaces of high strength PTFE sliding material.
Sealing elements	
7 strip seal 80	EPDM or chloroprene-rubber with high resistance to tear propagation, resistant to salt water, oil and ageing, available in any desired length. Hot vulcanization on site possible.
Anchorage elements	
8 carriageway anchors at the edge beams	flat and round steel made of S 235 JR G2
9 anchor studs at the support boxes	St37 K
10 support box	S 235 JR G2, to accommodate the sliding bearings, sliding springs, as well as providing the space required for the support bars in motion.

Functional Principles

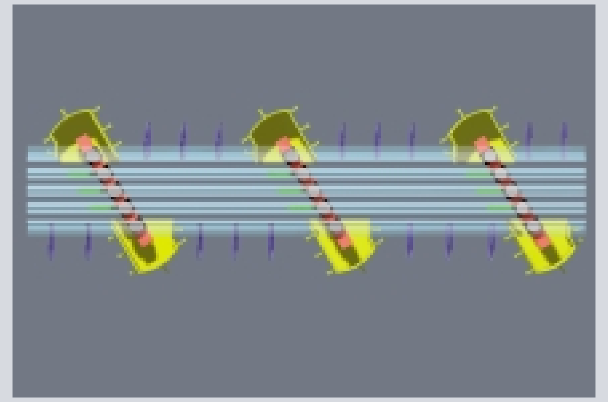
Type DS320
displacement of a support
bar that is fixed at one side
view from below



The centre beams of the Swivel-Joist Expansion Joint can slide on support bars with the help of sliding bearings. By means of the geometrical arrangement of the support bars the position of the centre beams is controlled such that the overall width of the joint opening is equally subdivided to the joint gaps between the centre beams and between the centre beams and edge beams respectively.

This both simple and effective control mechanism means an important advantage of the Swivel- Joist Expansion Joint. Unrestrained absorption of movements and simultaneous transmission of traffic loads is safeguarded without additional control elements and without any defined direction of movement.

In case of larger movements, in order to avoid large spans the support bars are arranged in parallel. In this case an additional restraint is required or the positioning of parallel support bars in the two neighbouring traffic directions must be arranged such that they are inclined to each other.



The resilient bearings in respect to torsion enable horizontal and also vertical displacements of the structure as well as differences in height of the joint edges in case of a longitudinal slope.

The ample space in the joist-boxes serves to accommodate the motion sequence of the swivelling support bars in motion. The total movement of a support bar can be allocated to the two edges of the joint arbitrarily. Quite frequently the movement of the support bar is absorbed at one side, for example at the abutment, whereas at the opposite edge the support bar can rotate but is fixed in its displacement.

It will also be possible that for geometrical reasons, e.g. because of prestressing cables, the one-side displaceable support bars can be arranged in an alternating way.

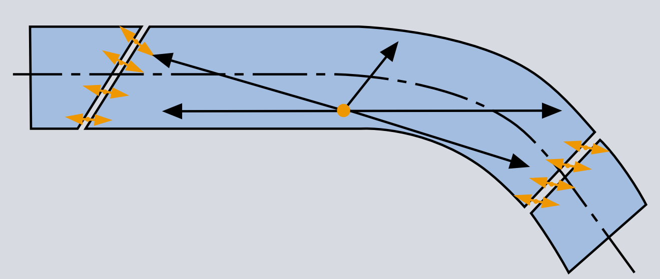
The total movement can be distributed to both edges of the joint as per requirement or desire, for instance in equal parts. In steel bridges the edge structure is supported on cantilevers or supporting girders parallel to the end cross girder. As a rule the cantilever plates that are fixed to the edge structure in the manufacturing site are then welded to the steel end cross girder.

In shifting the movement to the opposite abutment, the eccentricities of the traffic loads that are introduced can be reduced to a minimum.

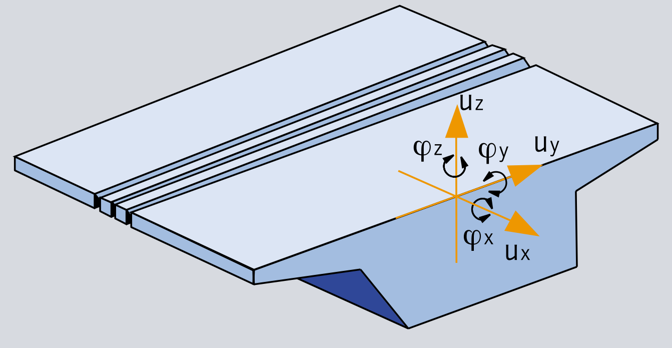
Contrary to the Girder Grid Joint, this type of Expansion Joint can accommodate the largest movements applied in bridge construction so far, which is facilitated by the fact that all centre beams are commonly supported by one support bar.

Versatility

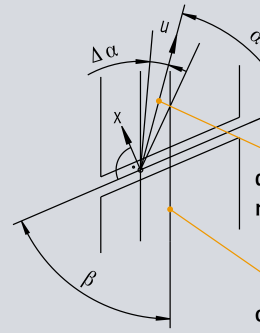
MAURER Swivel-Joist Expansion Joints can absorb all movements customary in bridge construction. The versatile mechanism of this type makes it suitable for variable skew angles along axes centred on a fixed point and also for complex hybrid movements and rotations about the spatial axes x, y and z.



The recess sizes given in the following pages will give the planner of the bridge structure some assistance for designing. The distribution of a support bar movement to each joint edge can be settled as per requirement or liking, other solutions than those shown below can be designed as well. All dimensions given are without obligation to the planner. For every project all dimensions will have to be determined case by case.



Geometrical restrictions which might be the consequence of the geometry of the boxes and support bars, can be changed by special design any time.



Due to the high standardisation expenditure resulting from Technical Test Specifications according to TL/TP-FÜ only applications of frequent use had been considered (please also refer to the appertaining documents). In Germany the admissible movement per joint gap in transverse direction to the joint-axis has been restricted to 65 mm. However, all expansion joints are designed to take movements of 80 mm. The following table shows the admissible movements for standard designs of the individual types.

type	weight [kg/m]	type	weight [kg/m]
DS160	270	DS720	930
DS240	350	DS800	1030
DS320	440	DS880	1140
DS400	530	DS960	1260
DS480	620	DS1040	1380
DS560	720	DS1120	1500
DS640	820	DS1200	1620

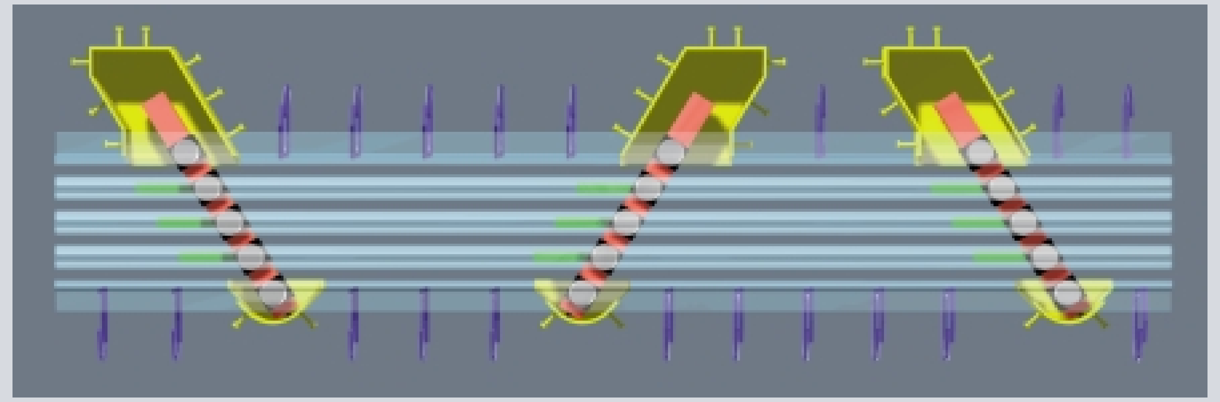
The given numbers of the weight of joints is only to calculate the size of cranes and lifting devices.

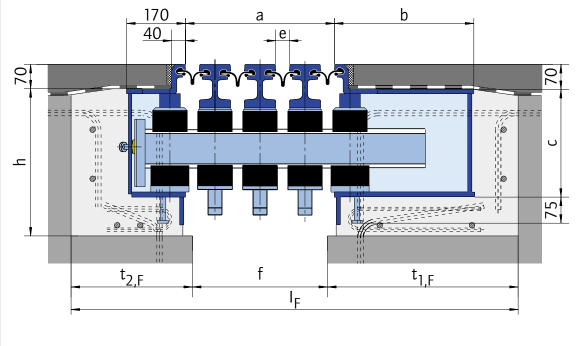
n	type	u_x [mm]	u_y *) [mm]	u_z *) [mm] mid-position	α [°]	Δα	β [°]
2	DS160	130 (160)	± 80	± 10			
3	DS240	195 (240)	± 120	± 15			
4	DS320	260 (320)	± 160	± 20			
5	DS400	325 (400)	± 200	± 25			
6	DS480	390 (480)	± 240	± 30			
7	DS560	455 (560)	± 280	± 35			
8	DS640	520 (640)	± 320	± 40	90° ± 45°	any	any
9	DS720	585 (720)	± 360	± 40			
10	DS800	650 (800)	± 400	± 40			
11	DS880	715 (880)	± 440	± 40			
12	DS960	780 (960)	± 480	± 45			
13	DS1040	845 (1040)	± 520	± 45			
14	DS1120	910 (1120)	± 560	± 45			
15	DS1200	975 (1200)	± 600	± 45			

*) Values apply to standard design, bigger values are possible, too.

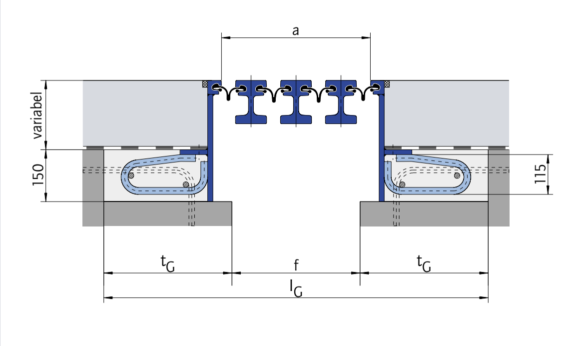
Movement of support bars to one side only

Recess dimensions

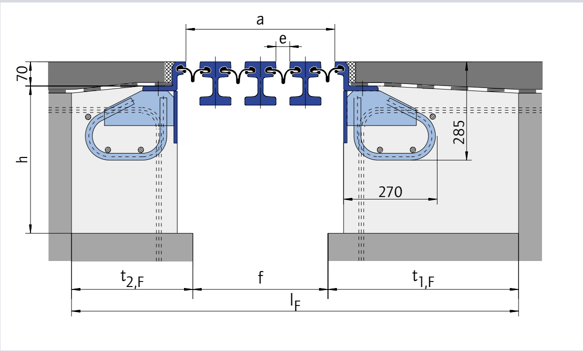




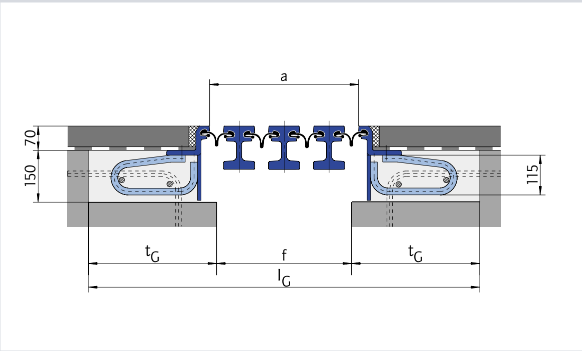
cross section through support box at carriageway



footway cross section – version 1



carriageway cross section at anchorage



footway cross section – version 2

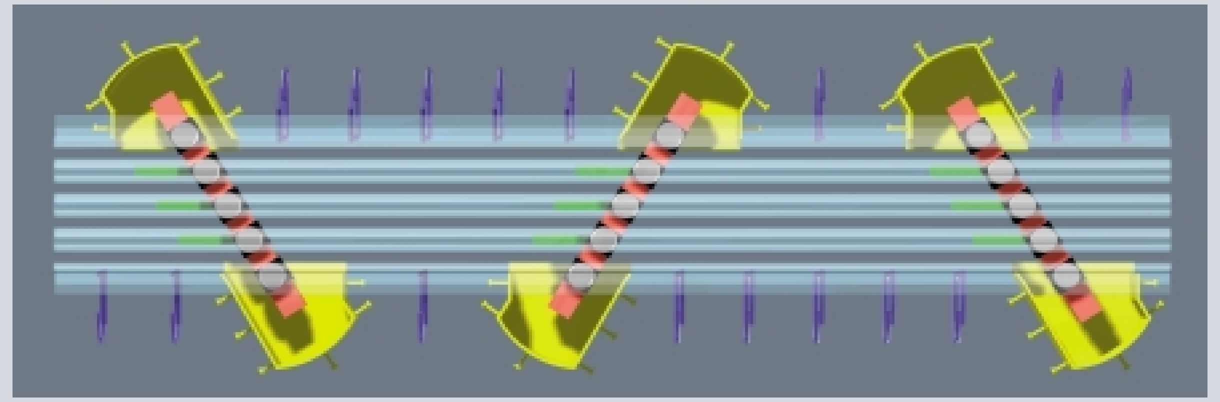
Presetting of gap dimension e = 30 mm											
MAURER exp. joint		structural dimensions			concrete recess dimensions			concrete gap dimensions			
n	type	a [mm]	b [mm]	c [mm]	h [mm]	$t_{1,F}$ [mm]	$t_{2,F}=t_G$ [mm]	f_{min} [mm]	f_{max} [mm]	l_F [mm]	l_G [mm]
2	DS160	150	260	290	420	400	350	115	130	865	815
3	DS240	270	310	300	430	450	380	225	250	1055	985
4	DS320	390	360	310	440	500	390	300	370	1190	1080
5	DS400	510	410	320	450	560	400	410	490	1370	1210
6	DS480	630	460	330	460	620	410	520	610	1550	1340
7	DS560	750	510	340	470	680	420	630	730	1730	1470
8	DS640	870	560	350	480	740	430	740	850	1910	1600
9	DS720	990	610	360	490	800	440	850	970	2090	1730
10	DS800	1110	660	370	500	860	450	960	1090	2270	1860
11	DS880	1230	710	380	510	920	460	1070	1210	2450	1990
12	DS960	1350	760	390	520	980	470	1180	1330	2630	2120
13	DS1040	1470	810	400	530	1040	480	1290	1450	2810	2250
14	DS1120	1590	860	410	540	1100	490	1400	1570	2990	2380
15	DS1200	1710	910	420	550	1160	500	1510	1690	3170	2510

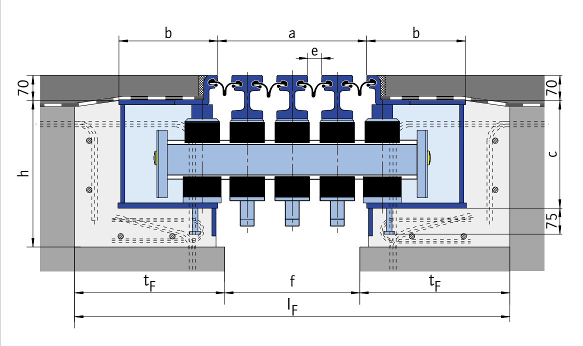
For structures acc. to directives TL/TP-FÜ you must also observe the details given in the standard testing specifications

- All dimensions are rectangular to the joint axis y.
- n = number of sealing elements
- a, f and l apply to a presetting dimension e = 30 mm for every joint gap and must be adjusted by n x Δe in case of deviating presetting dimension e.
- recesses for footway joists, restraints and tube openings as a rule require prior consent between the planner of the structure and the manufacturer of the expansion joint.
- Smaller recess dimensions are possible by special design of MAURER.

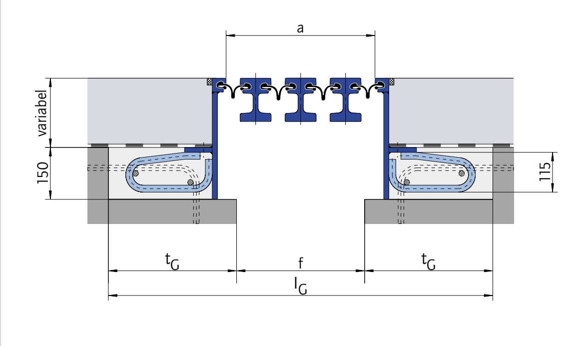
Movement of support bars to both sides

Recess dimensions

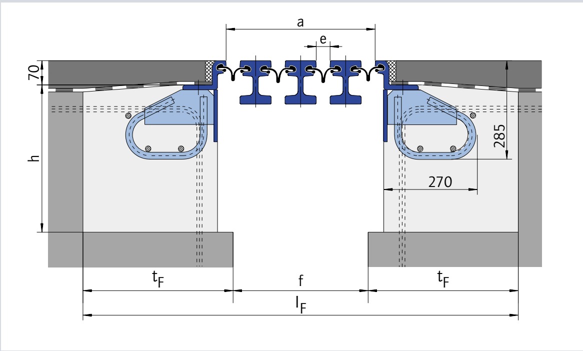




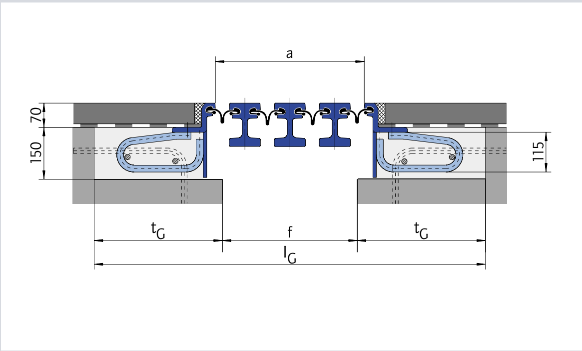
cross section through support box at carriageway



footway cross section – version 1



carriageway cross section at anchorage



footway cross section – version 2

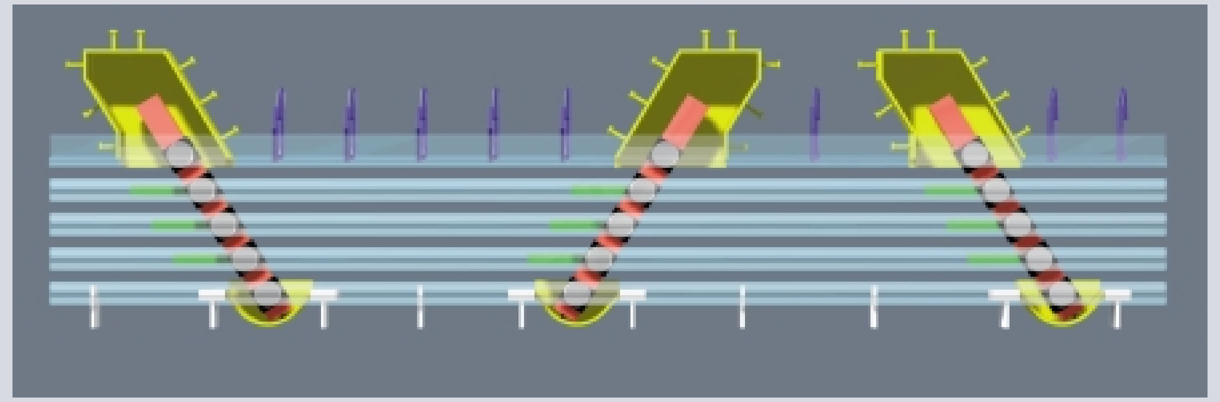
Presetting of gap dimension e = 30 mm											
MAURER exp. joint		design data			concrete-recess dimensions			concrete-gap dimensions			
n	type	a [mm]	b [mm]	c [mm]	h [mm]	t_F [mm]	t_G [mm]	f_{min} [mm]	f_{max} [mm]	l_F [mm]	l_G [mm]
2	DS160	150	215	290	420	350	350	115	130	815	815
3	DS240	270	255	300	430	395	380	225	250	1015	985
4	DS320	390	285	310	440	435	390	300	370	1170	1080
5	DS400	510	355	320	450	510	400	410	490	1430	1210
6	DS480	630	380	330	460	550	410	520	610	1620	1340
7	DS560	750	410	340	470	590	420	630	730	1810	1470
8	DS640	870	430	350	480	620	430	740	850	1980	1600
9	DS720	990	460	360	490	660	440	850	970	2170	1730
10	DS800	1110	490	370	500	690	450	960	1090	2340	1860
11	DS880	1230	515	380	510	730	460	1070	1210	2530	1990
12	DS960	1350	550	390	520	770	470	1180	1330	2720	2120
13	DS1040	1470	585	400	530	820	480	1290	1450	2930	2250
14	DS1120	1590	615	410	540	860	490	1400	1570	3120	2380
15	DS1200	1710	645	420	550	900	500	1510	1690	3310	2510

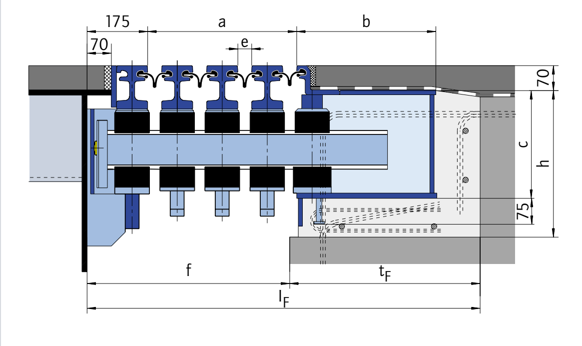
For structures acc. to directives TL/TP-FÜ you must also observe the details given in the standard testing specifications

- All dimensions are rectangular to the joint axis y.
- n = number of sealing elements
- a, f and l apply to a presetting dimension e = 30 mm for every joint gap and must be adjusted by n x Δe in case of deviating presetting dimension e.
- recesses for footway joists, restraints and tube openings as a rule require prior consent between the planner of the structure and the manufacturer of the expansion joint.
- Smaller recess dimensions are possible by special design of MAURER.

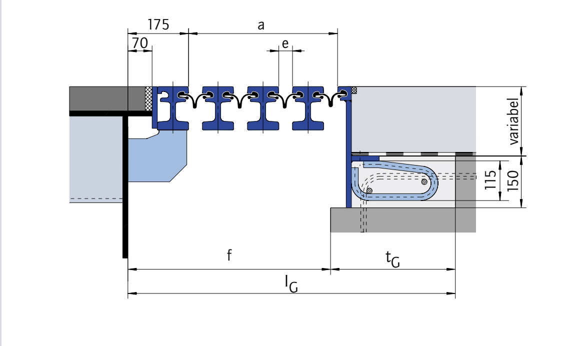
Steel connection

Recess dimensions

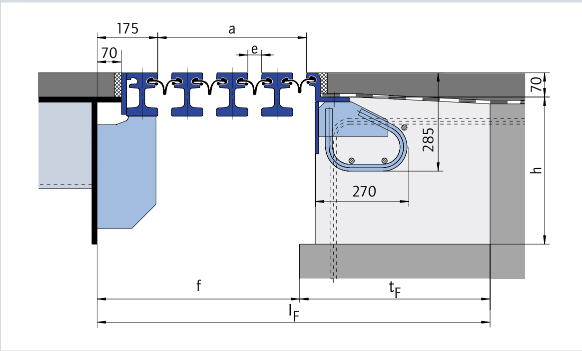




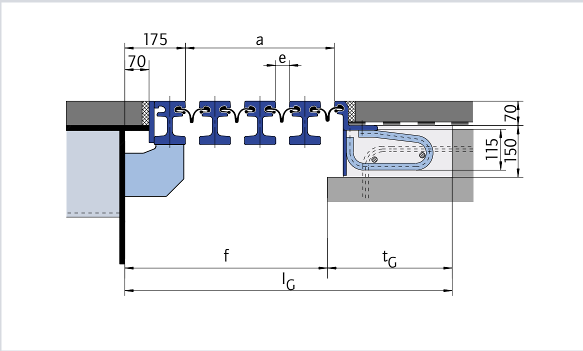
cross section through support box at carriageway



footway cross section – version 1



carriageway cross section at anchorage



footway cross section – version 2

Presetting of gap dimension e = 30 mm											
MAURER exp. joint		structural dimensions			concrete recess dimensions			concrete gap dimensions			
n	type	a [mm]	b [mm]	c [mm]	h [mm]	t_F [mm]	t_G [mm]	f_{min} [mm]	f_{max} [mm]	l_F [mm]	l_G [mm]
2	DS160	150	260	290	420	400	385	300	310	700	685
3	DS240	270	310	300	430	470	400	350	430	820	750
4	DS320	390	360	310	440	540	410	460	550	1000	870
5	DS400	510	410	320	450	610	425	570	670	1180	995
6	DS480	630	460	330	460	680	440	680	790	1360	1120
7	DS560	750	510	340	470	750	450	790	910	1540	1240
8	DS640	870	560	350	480	820	470	900	1030	1720	1370
9	DS720	990	610	360	490	890	480	1010	1150	1900	1490
10	DS800	1110	660	370	500	960	500	1120	1270	2080	1620
11	DS880	1230	710	380	510	1030	520	1230	1390	2260	1750
12	DS960	1350	760	390	520	1100	530	1340	1510	2440	1870
13	DS1040	1470	810	400	530	1170	550	1450	1630	2620	2000
14	DS1120	1590	860	410	540	1240	560	1560	1750	2800	2120
15	DS1200	1710	910	420	550	1310	570	1670	1870	2980	2240

For structures acc. to directives TL/TP-FÜ you must also observe the details given in the standard testing specifications

- All dimensions are rectangular to the joint axis y.
- n = number of sealing elements
- a, f and l apply to a presetting dimension e = 30 mm for every joint gap and must be adjusted by n x Δe in case of deviating presetting dimension e.
- recesses for footway joists, restraints and tube openings as a rule require prior consent between the planner of the structure and the manufacturer of the expansion joint.
- Smaller recess dimensions are possible by special design of MAURER.

Bridges with MAURER Swivel-Joist Expansion Joints

Vasco da Gama Bridge, Portugal
with fuse box for
earthquake movements
built: 1997
Cable-stayed bridge
main span: 829 m
type of joint:
DS1440 59.00 lin. metres

Storebælt East Bridge, Denmark
built: 1996
Suspension bridge
main span: 1624 m
type of joint:

DS2000	51.40 lin. metres
DS1520	25.70 lin. metres
DS1200	25.70 lin. metres
DS960	25.70 lin. metres
DS800	25.70 lin. metres

Höga Kusten Bridge, Sweden
built: 1997
suspension bridge
main span: 1210 m
type of joint:
DS1840 36.80 lin. metres

Stura di Demonte, Italy
built: 1999
Composite steel bridge
length of bridge: 2750 m
type of joint:
DS1200 24.50 lin. metres

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e-mail ba@mchn.maurer-soehne.de
Internet www.maurer-soehne.de

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P.O. Box 63 40, D-44520 Lünen
Phone ++49/2 31/ 4 34 01-0
Fax ++49/2 31/4 34 01-11

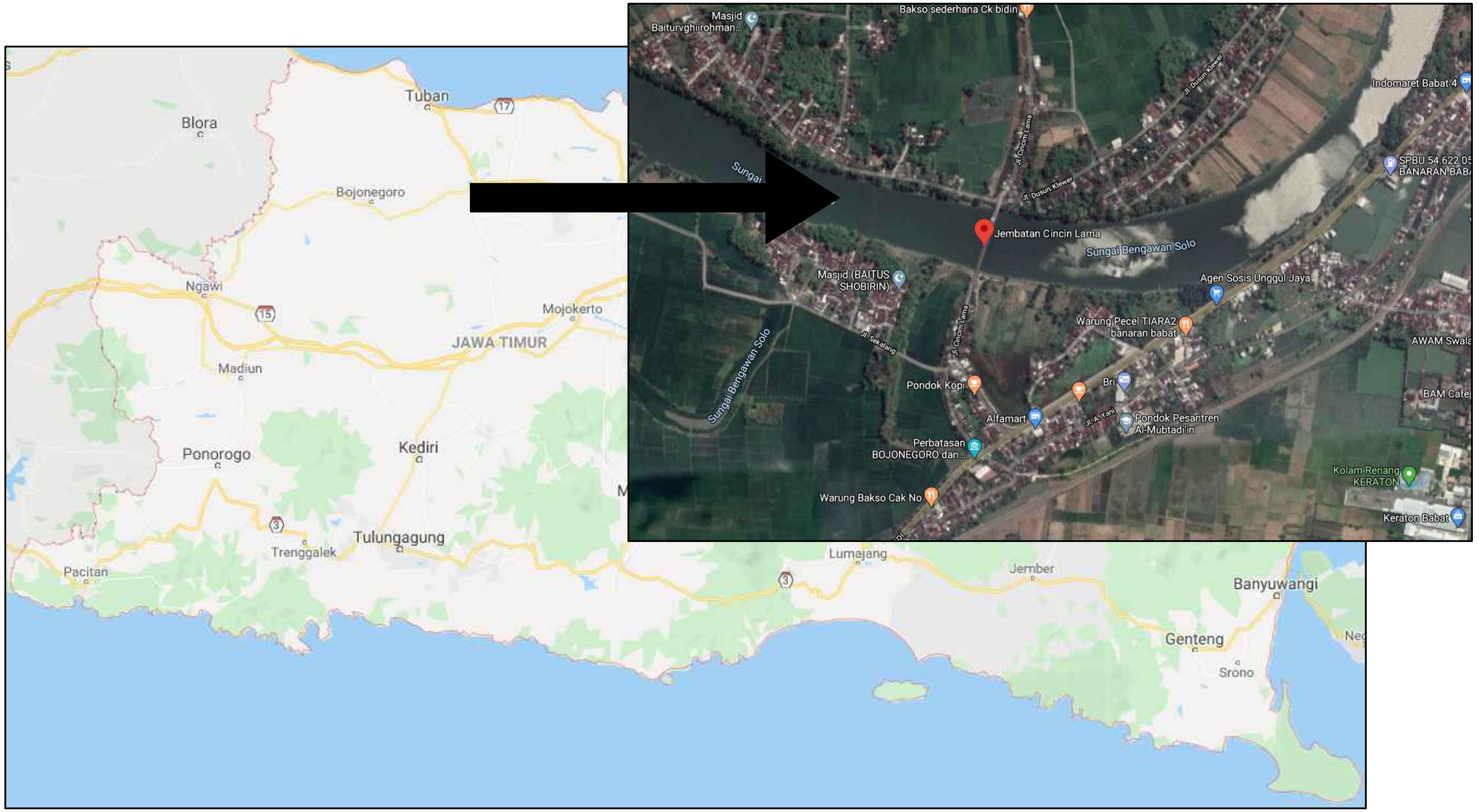
Maurer Söhne Subsidiary Plant
Kamenzer Str. 4–6, D-02994 Bernsdorf
P.O. Box 55, D-02992 Bernsdorf
Phone ++49/3 57 23/ 2 37-0
Fax ++49/3 57 23/2 37-20

BA17D-5000-11.00

Bridge Accessories

MAURER Swivel-Joist Expansion Joint

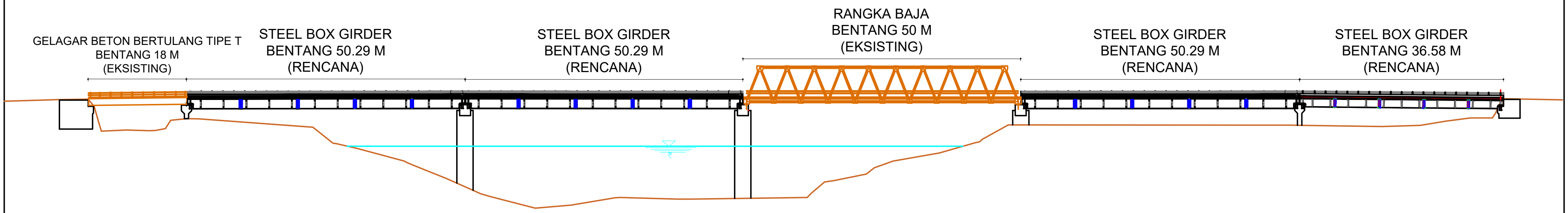
MAURER SÖHNE
Innovations in steel
Since 1876



Peta Lokasi

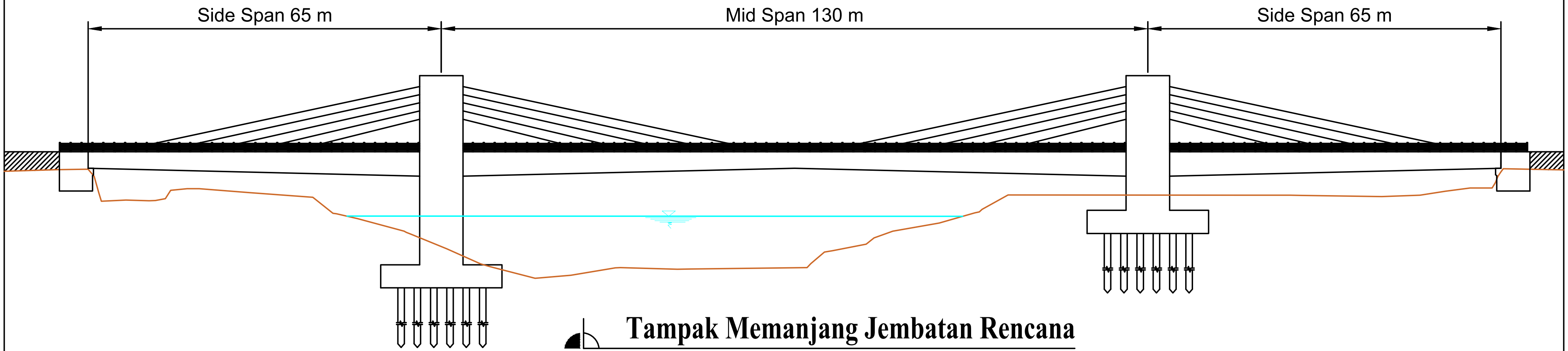
SKALA NTS

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Peta Lokasi	01	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Tampak Memanjang Jembatan Eksisting

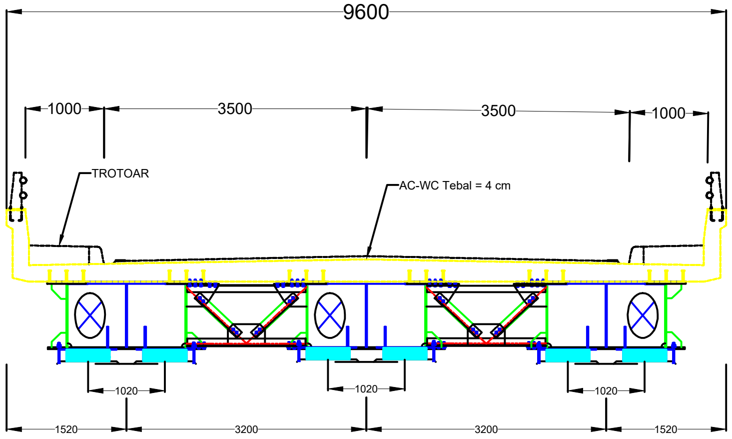
SKALA 1 : 1000



Tampak Memanjang Jembatan Rencana

SKALA 1 : 1000

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Tampak Memanjang Jembatan	02	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Tampak Melintang Jembatan Eksisting

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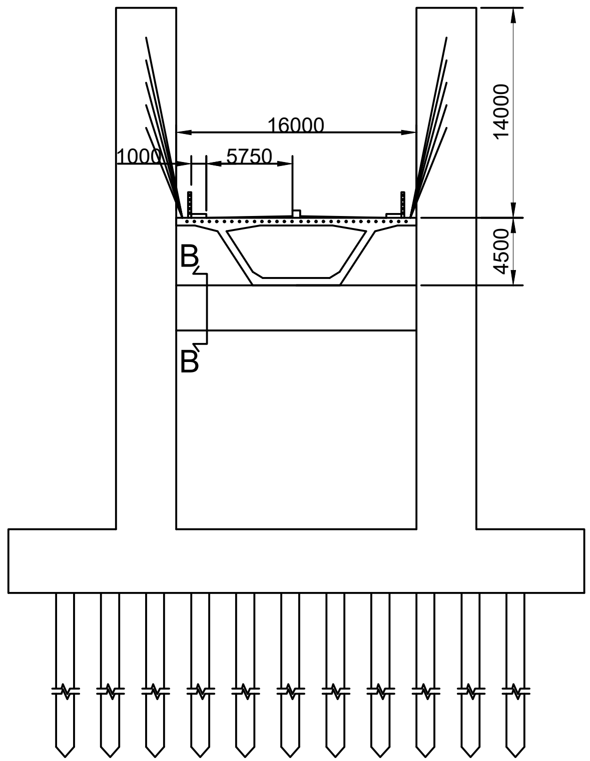
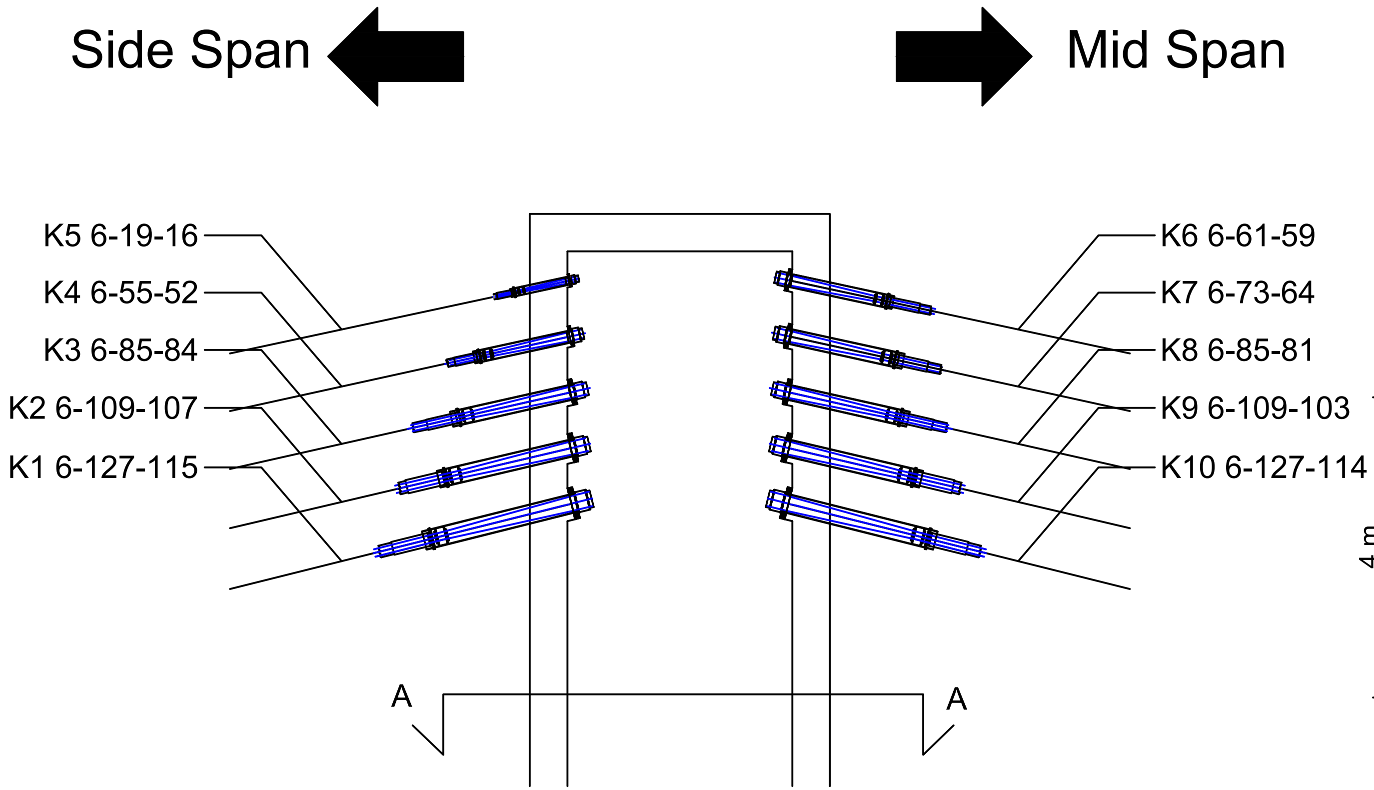


Tampak Melintang Jembatan Rencana

SKALA 1 : 500

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Tampak Melintang Jembatan	03	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Potongan Samping Pylon

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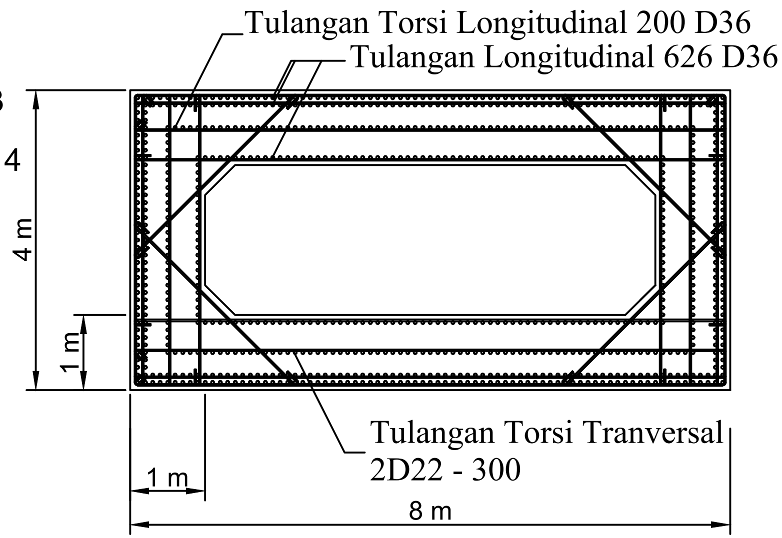
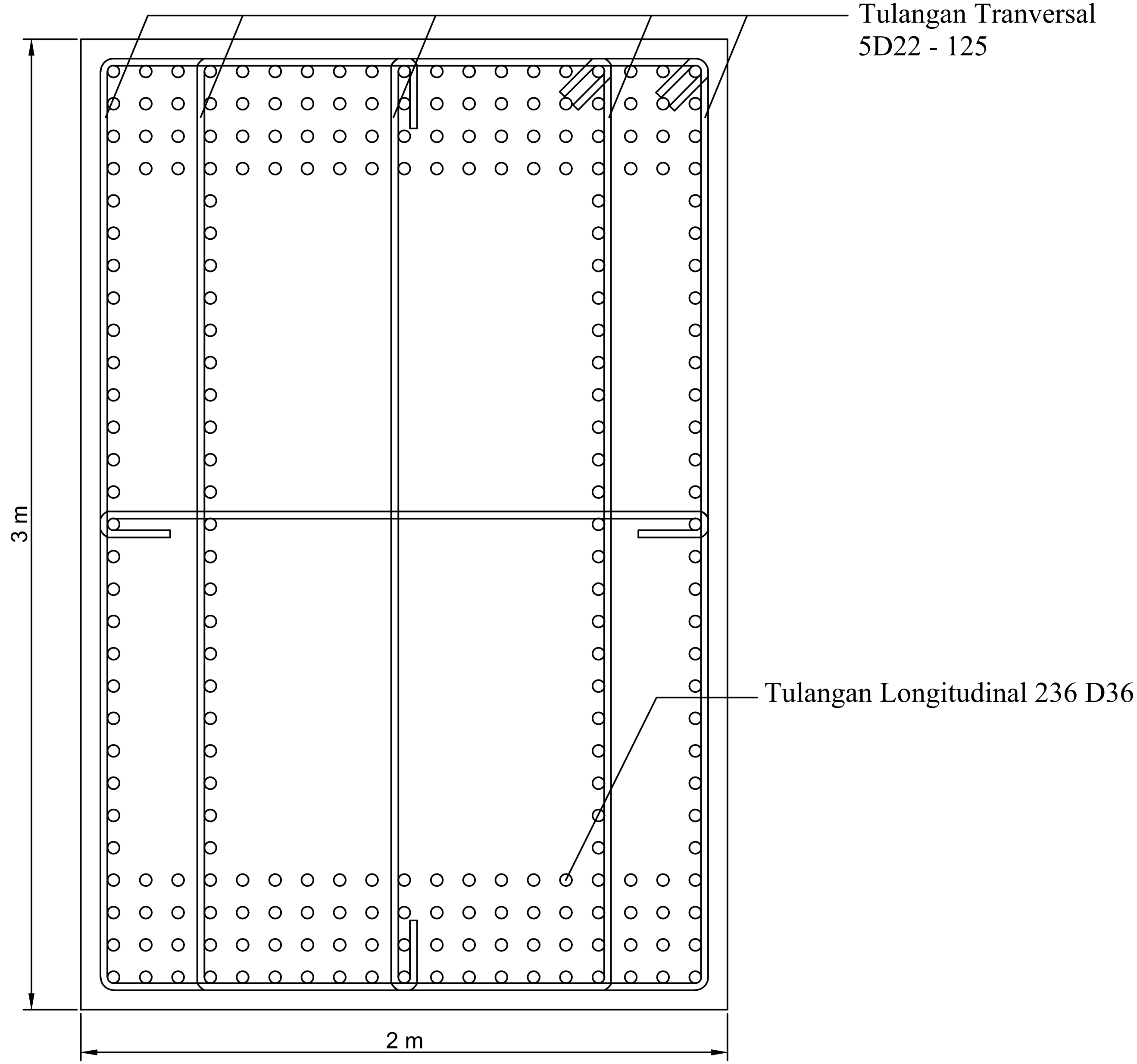


Potongan A-A Pylon

SKALA 1 : 100

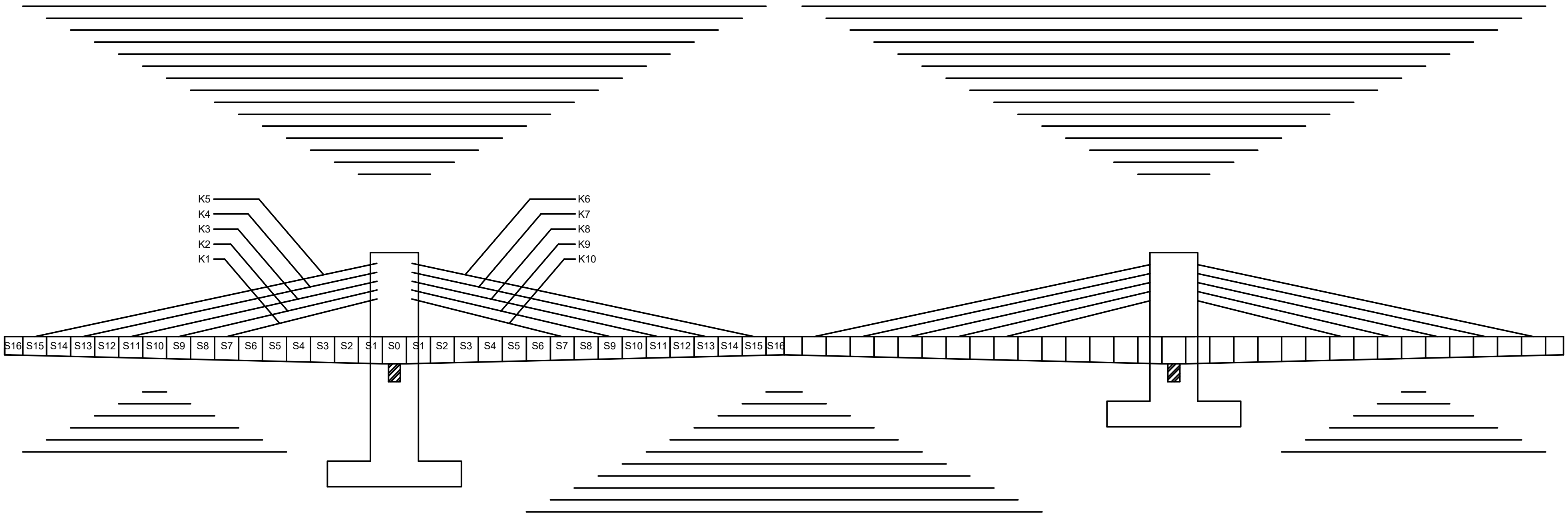
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Pylon	04	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Potongan B-B Balok

SKALA 1 : 20

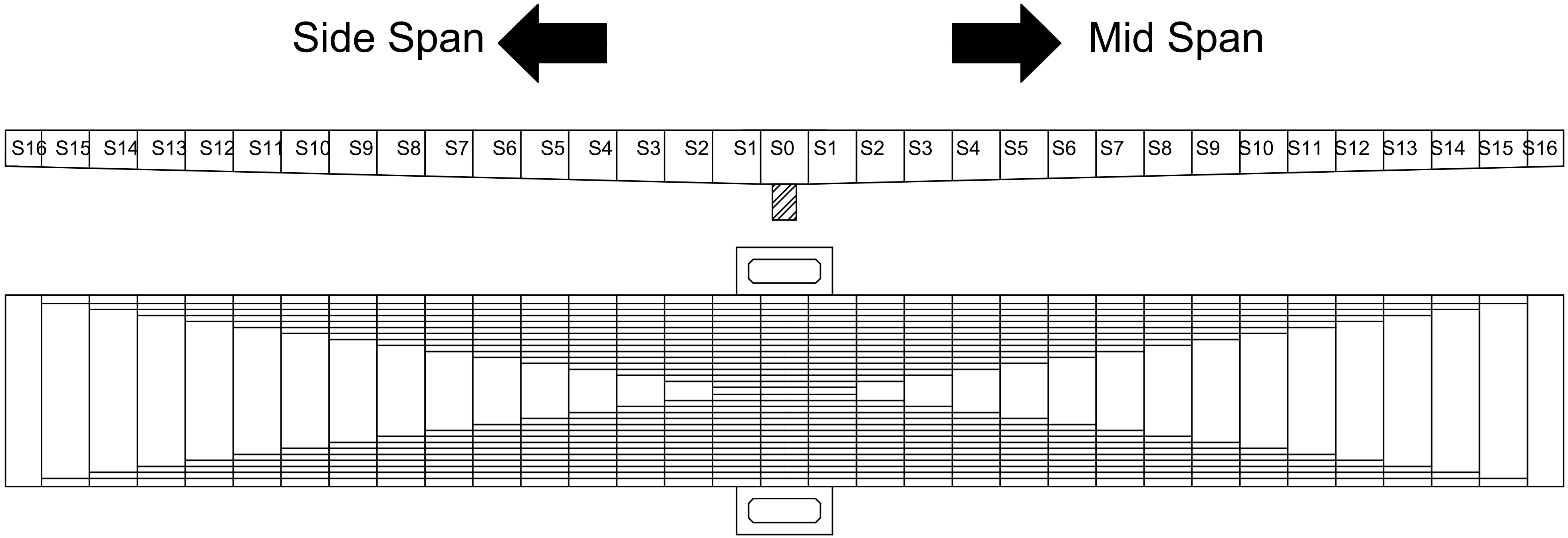
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Balok	05	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Layout Tendon

SKALA 1 : 1000

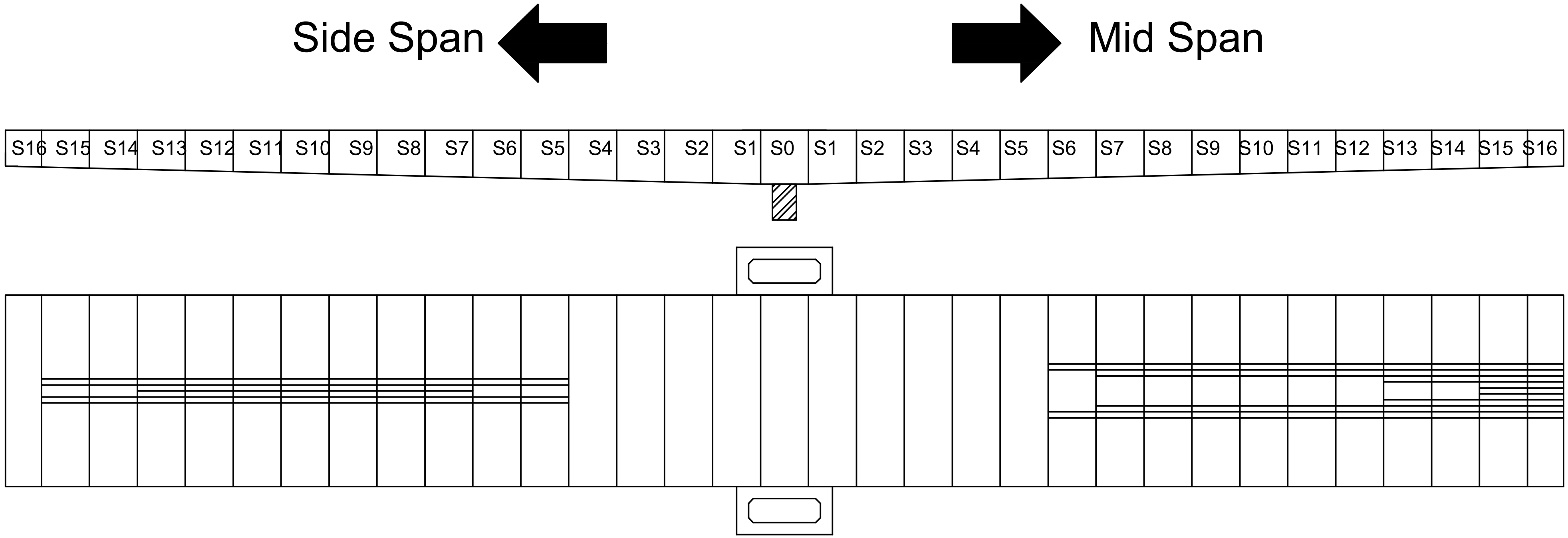
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Layout Tendon	**06**	**63**	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Layout Tendon Konstruksi

SKALA 1 : 500

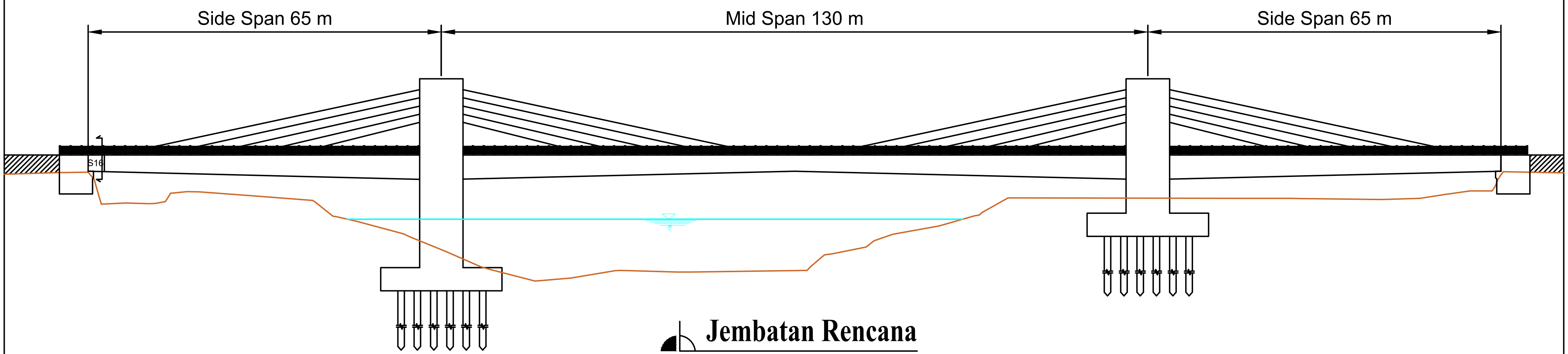
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Tendon Konstruksi	07	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Layout Tendon Menerus

SKALA 1 : 500

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Tendon Menerus	**08**	**63**	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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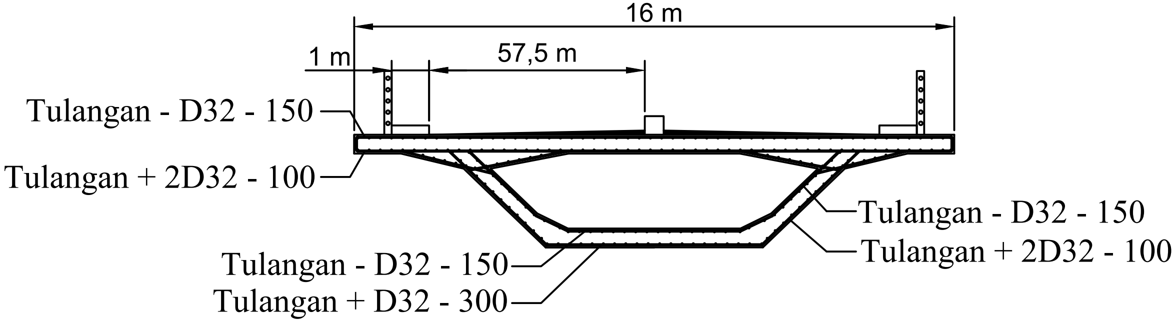
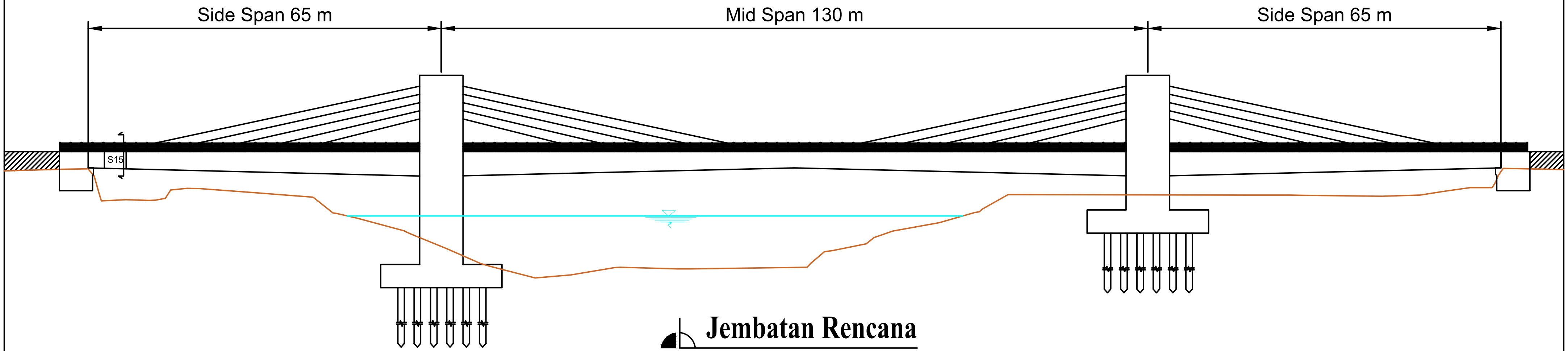


Potongan S16 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	09	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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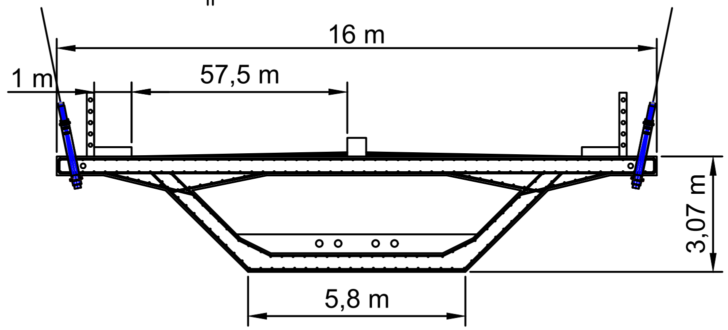
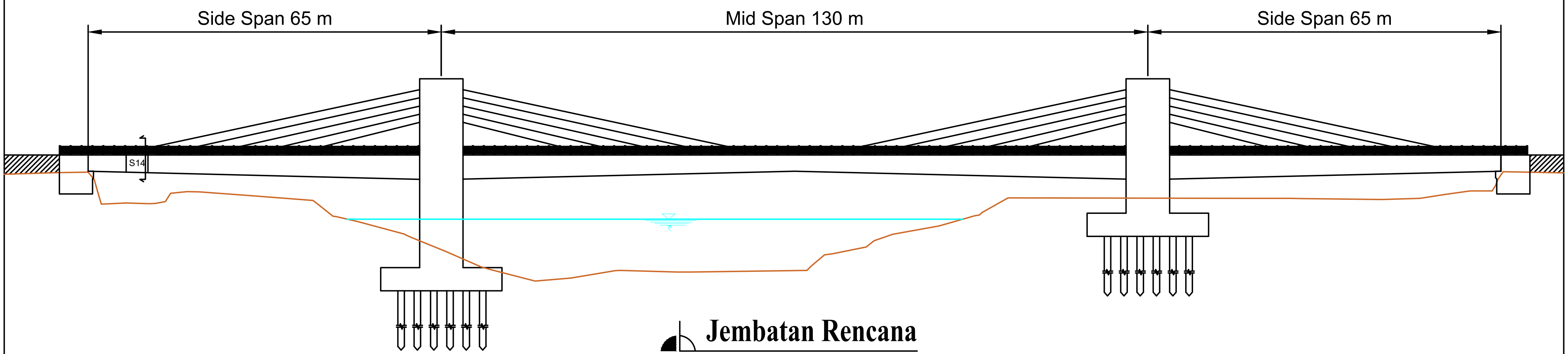


Potongan S15 Side Span

SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	10	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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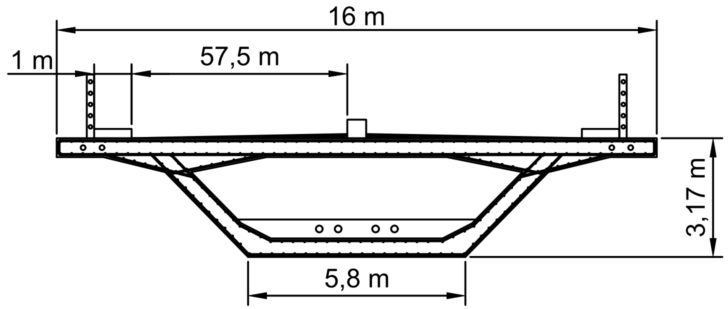
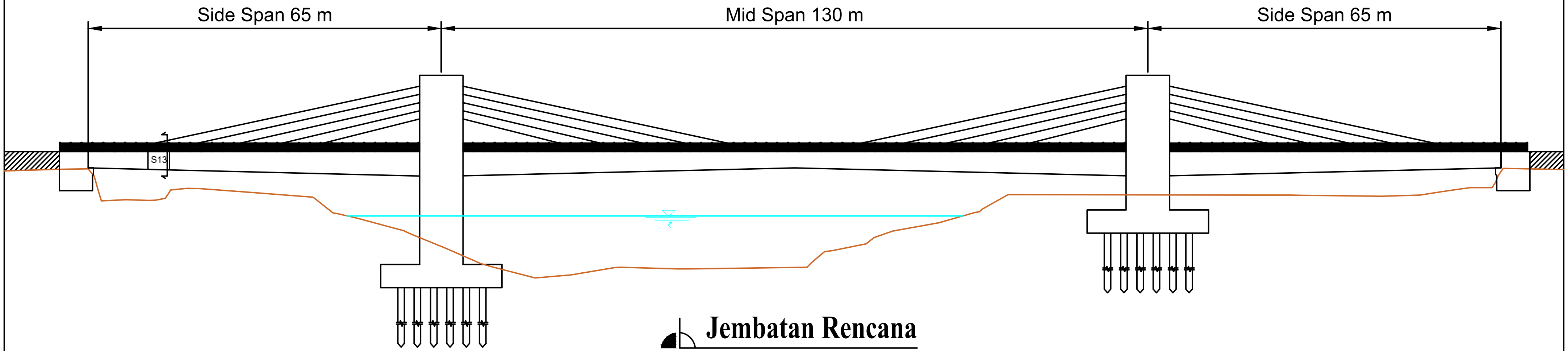


Potongan S14 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	11	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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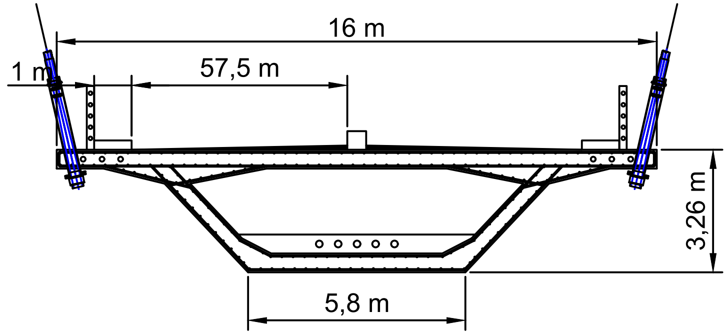
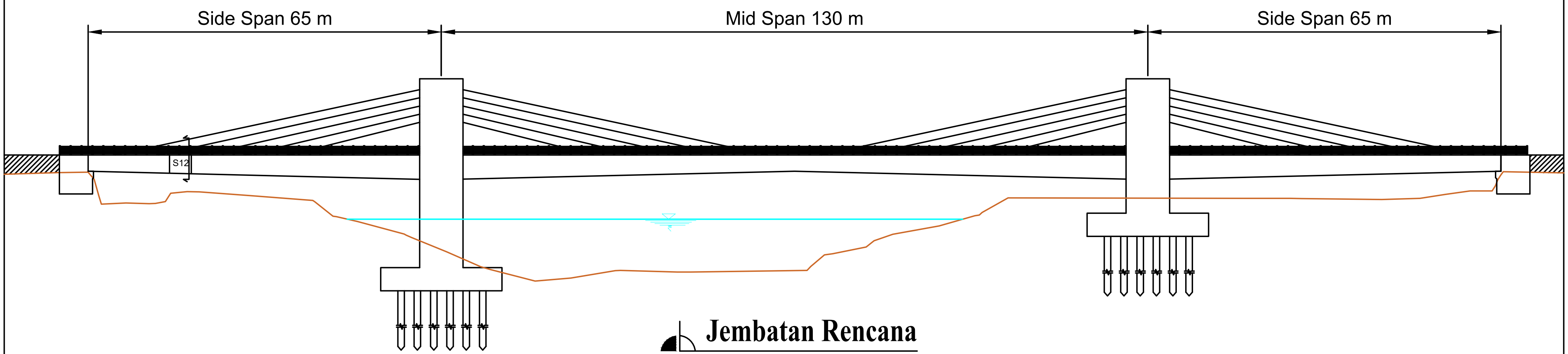


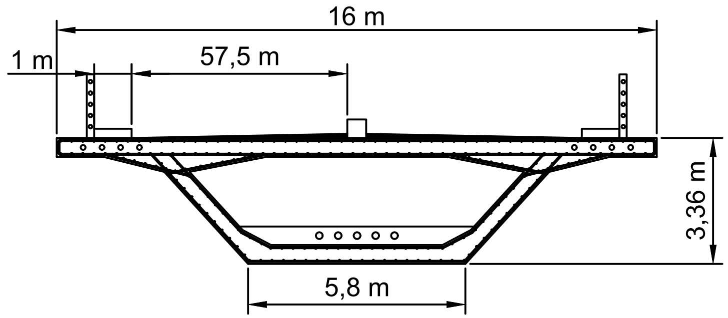
Potongan S13 Side Span

SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	12	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084

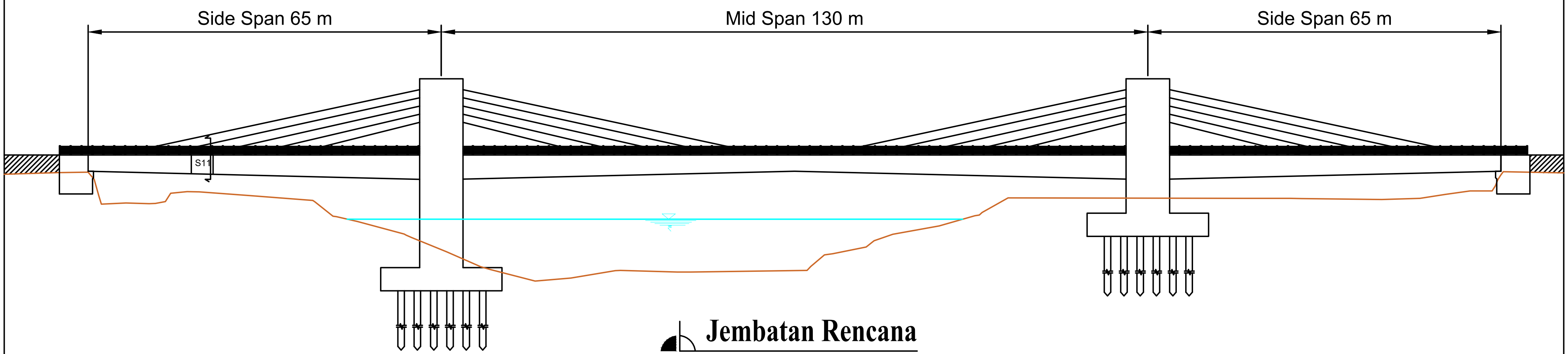


Jembatan Rencana
SKALA 1 : 1000



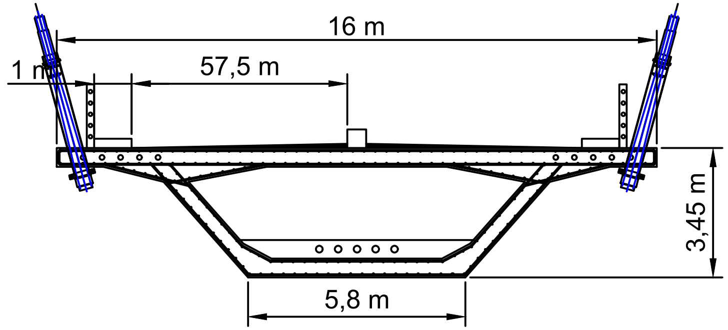
Potongan S12 Side Span
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	13	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

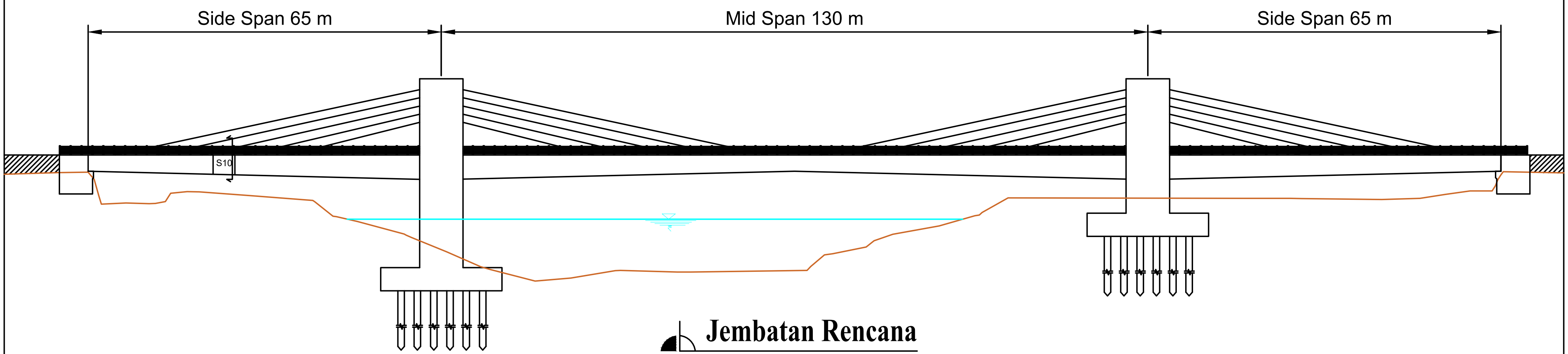
SKALA 1 : 1000



Potongan S11 Side Span

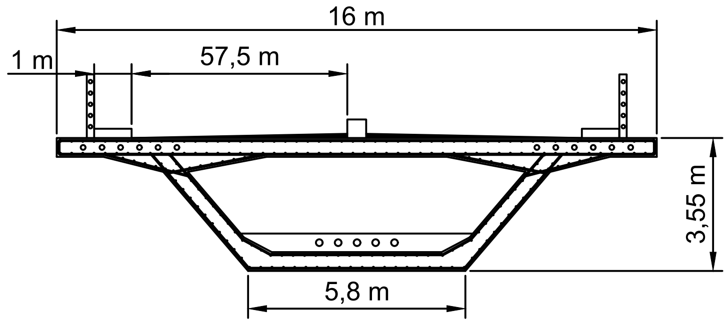
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	14	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

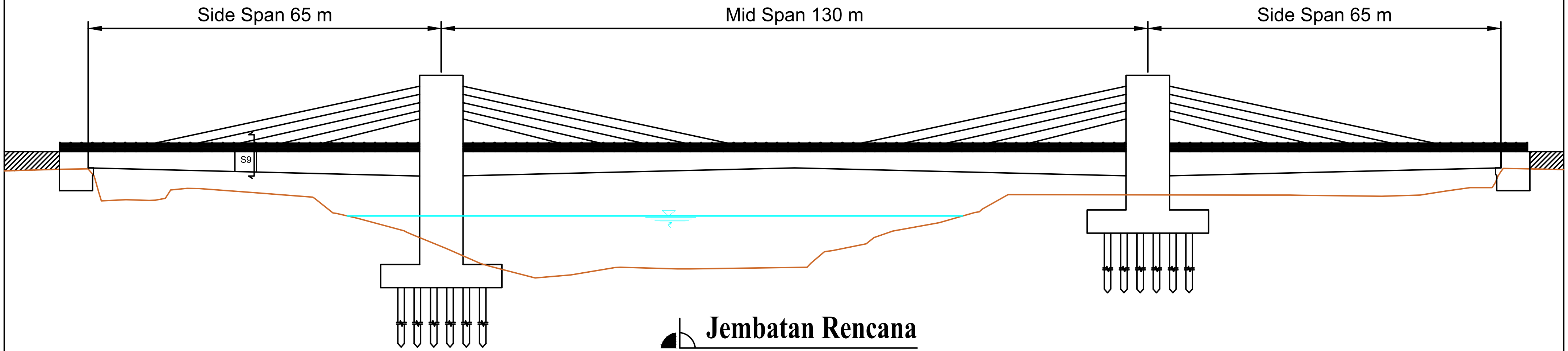
SKALA 1 : 1000



Potongan S10 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	15	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana
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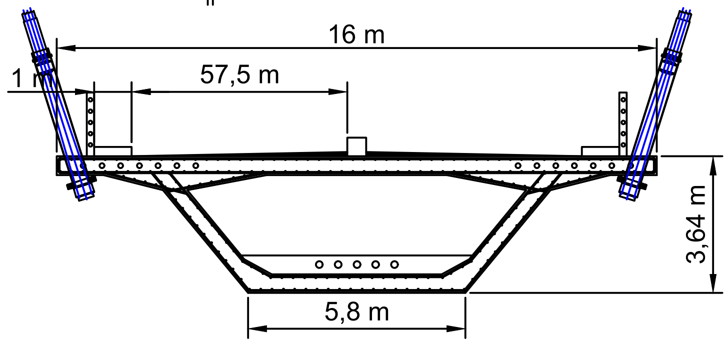
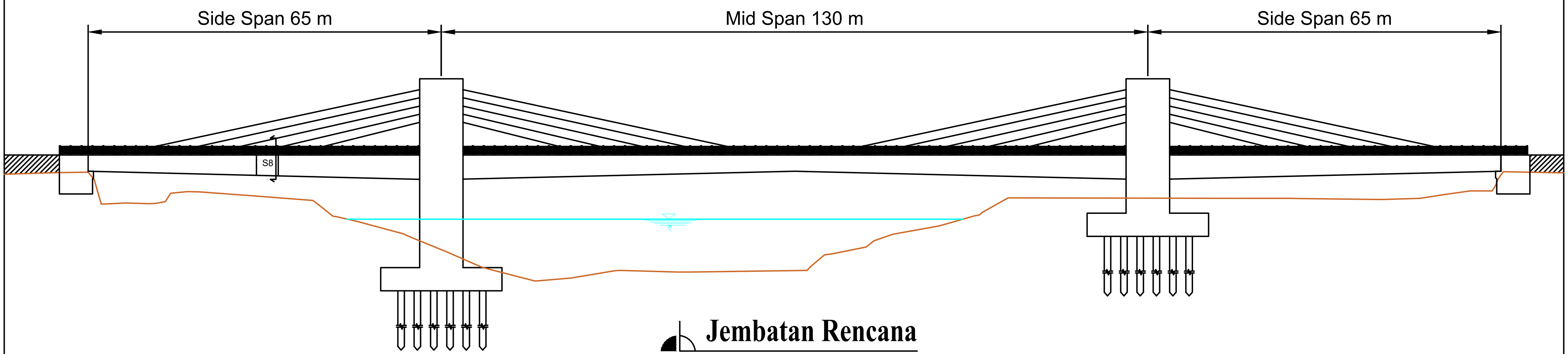
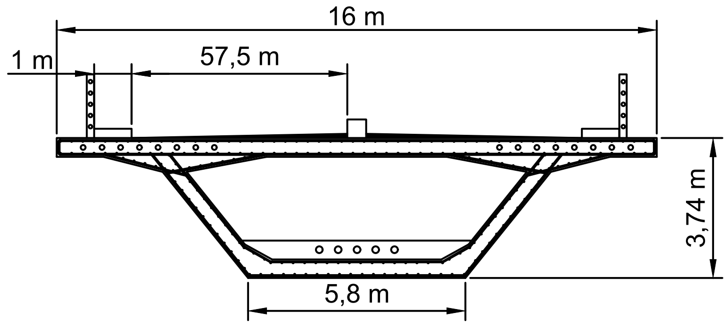


Potongan S9 Side Span
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	16	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084

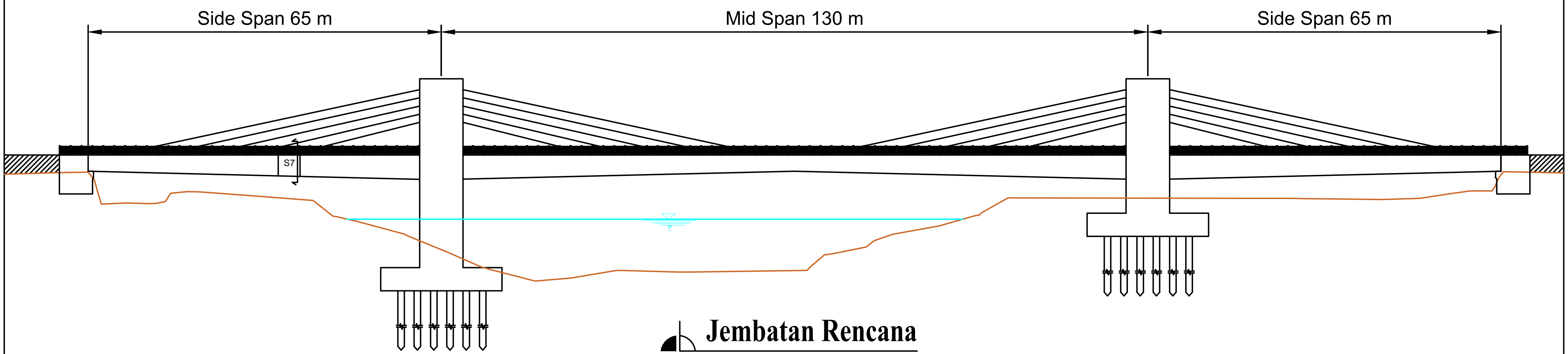


Jembatan Rencana
SKALA 1 : 1000



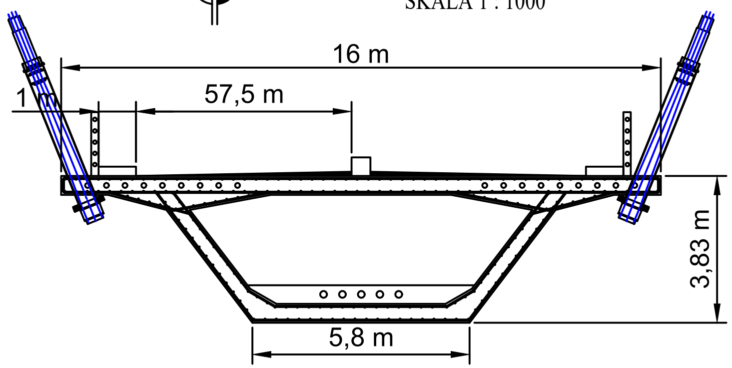
Potongan S8 Side Span
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	17	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

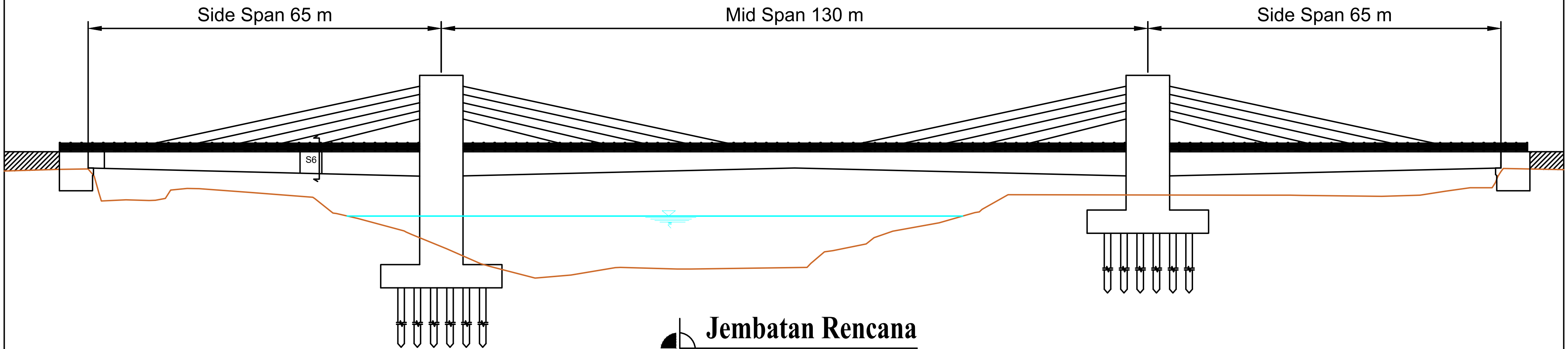
SKALA 1 : 1000



Potongan S7 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	18	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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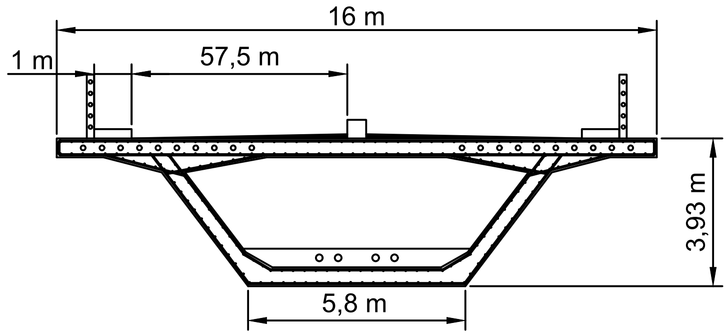
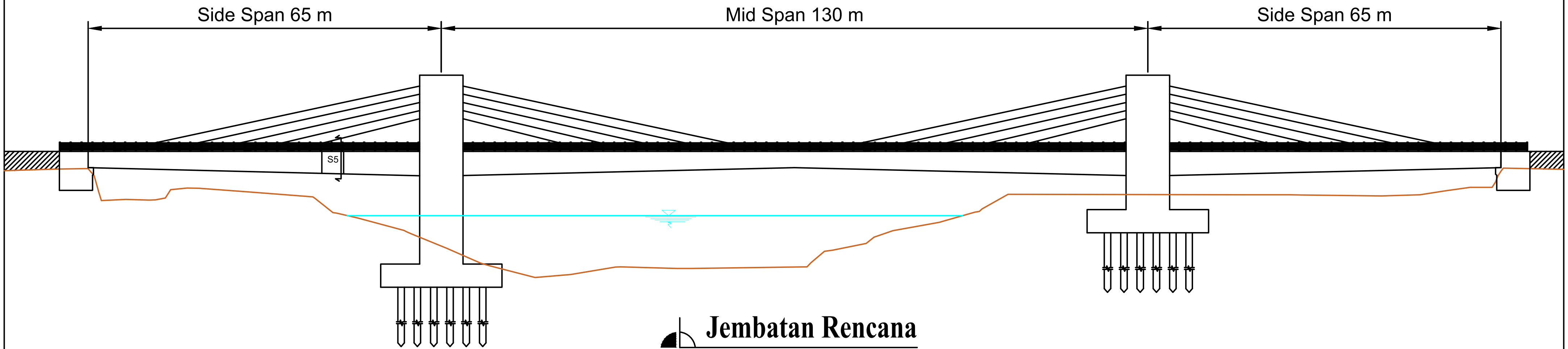


Potongan S6 Side Span

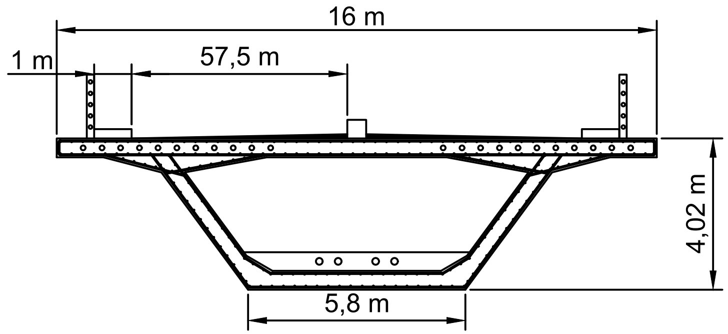
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	19	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

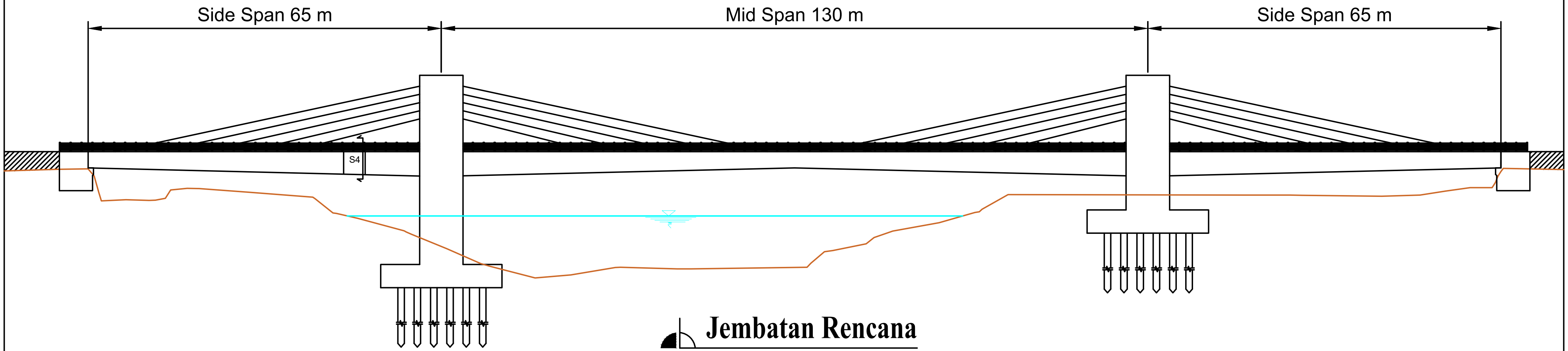
SKALA 1 : 1000



Potongan S5 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	20	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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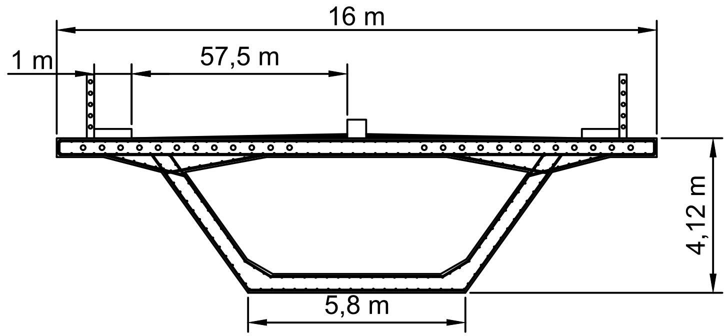
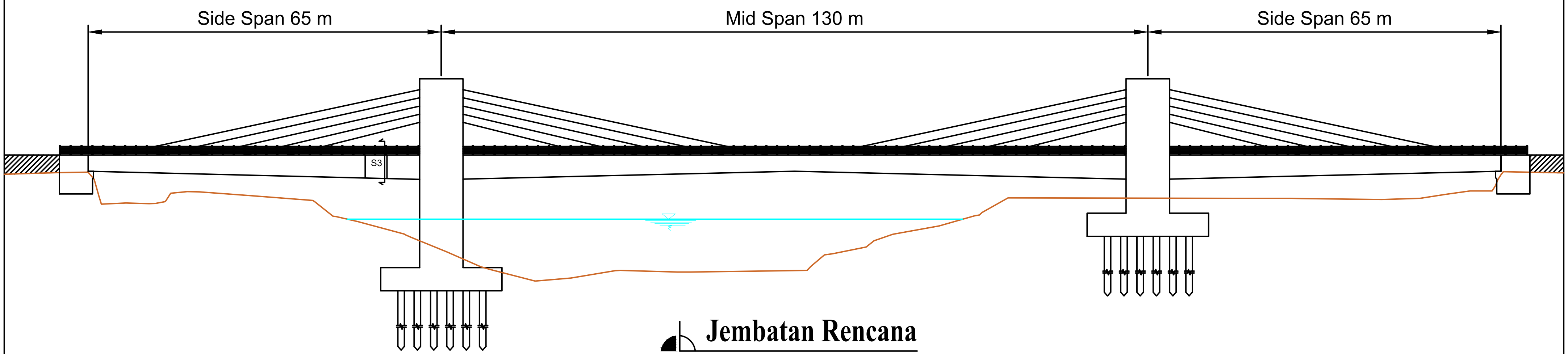


Potongan S4 Side Span

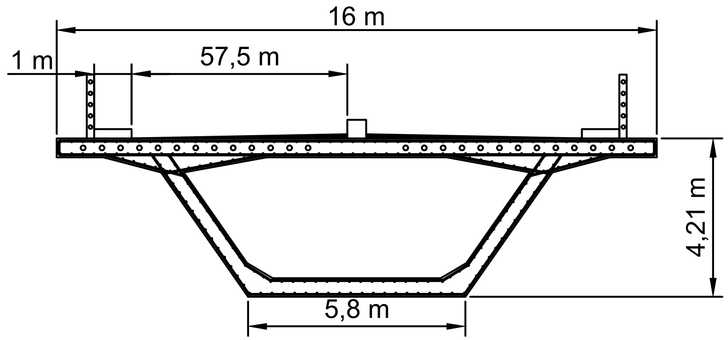
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	21	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

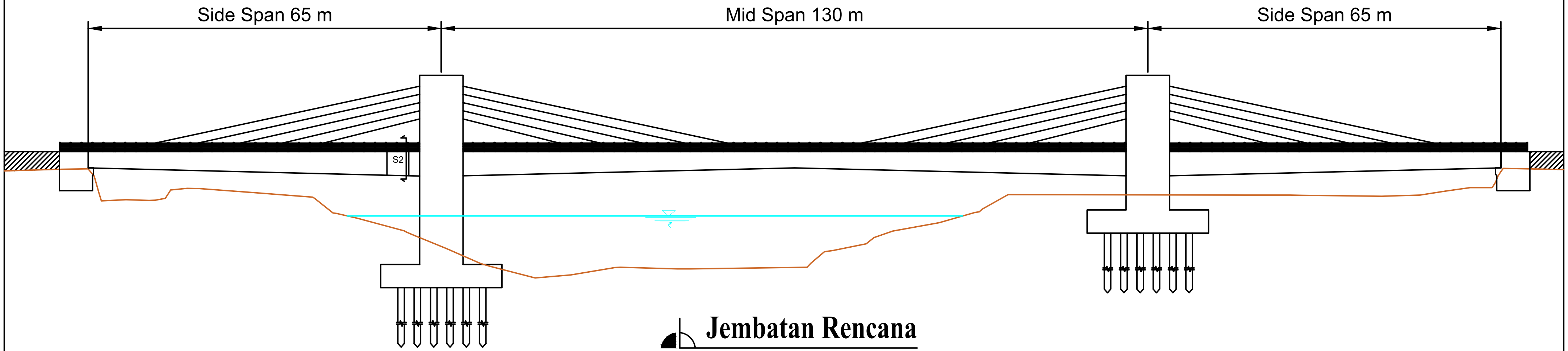
SKALA 1 : 1000



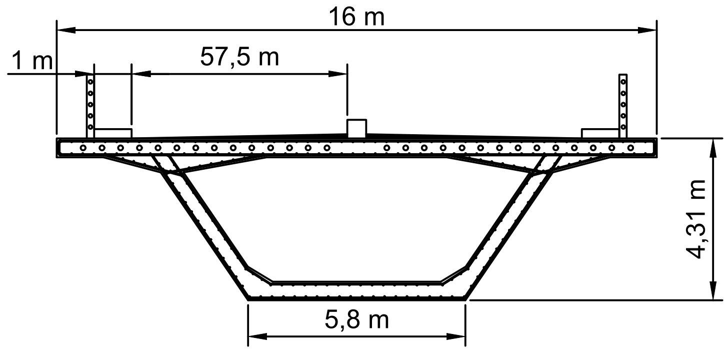
Potongan S3 Side Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	22	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084

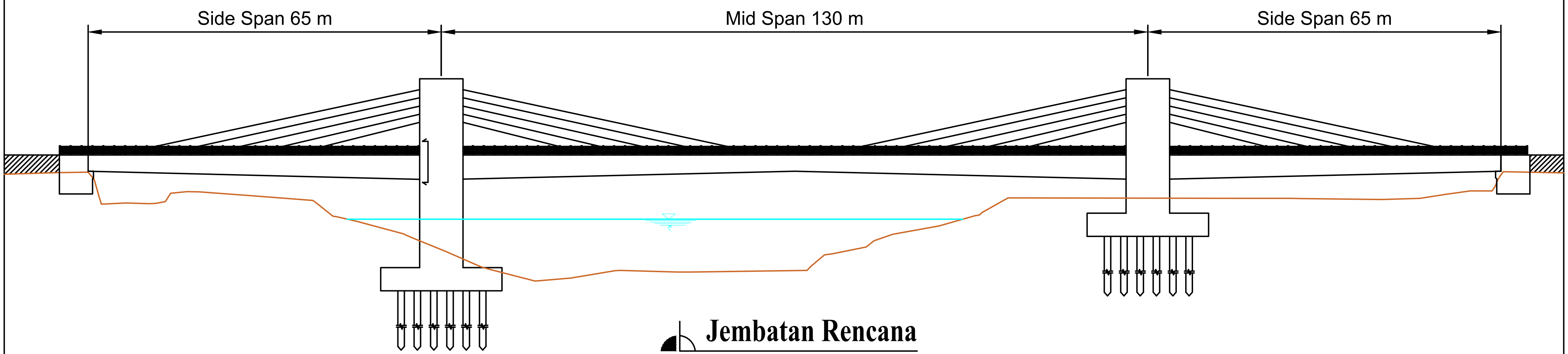


Jembatan Rencana
SKALA 1 : 1000



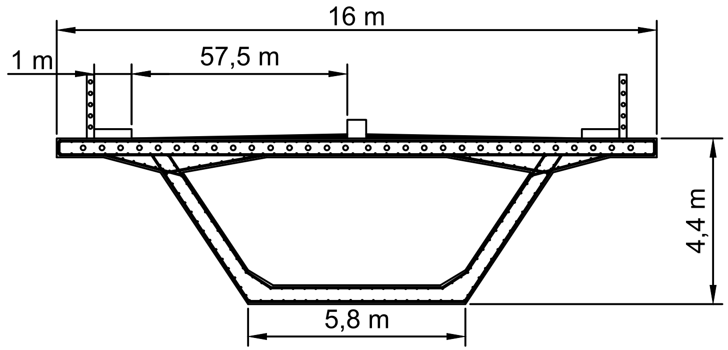
Potongan S2 Side Span
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	23	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

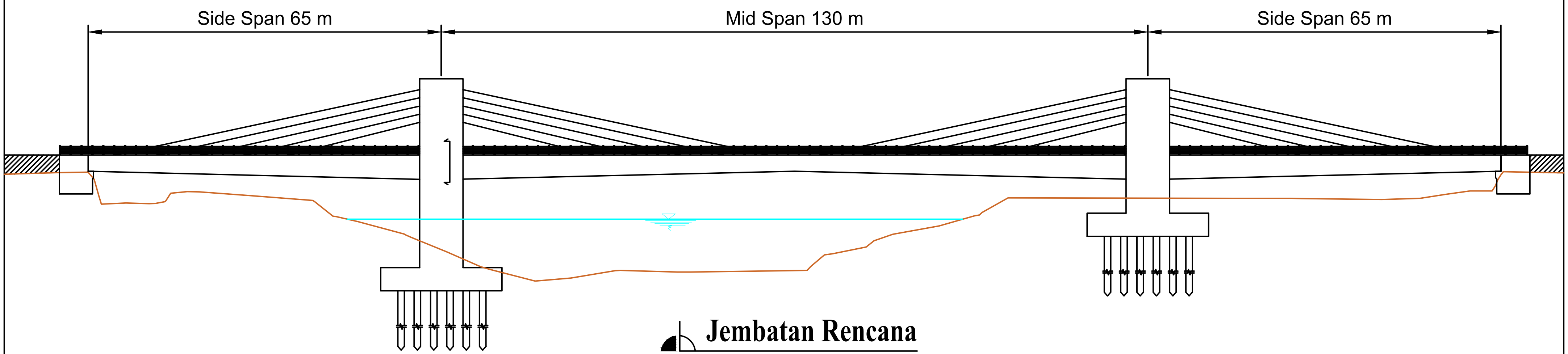
SKALA 1 : 1000



Potongan S1 Side Span

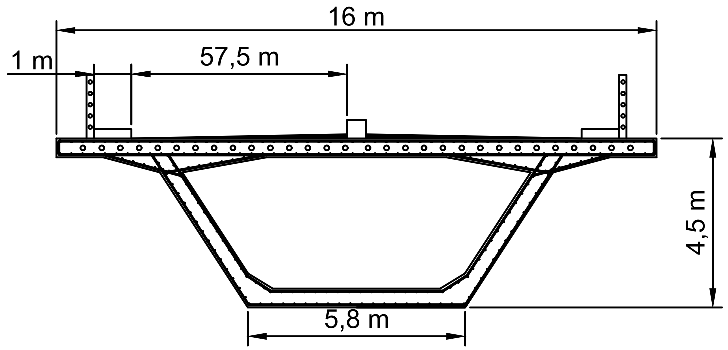
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	24	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

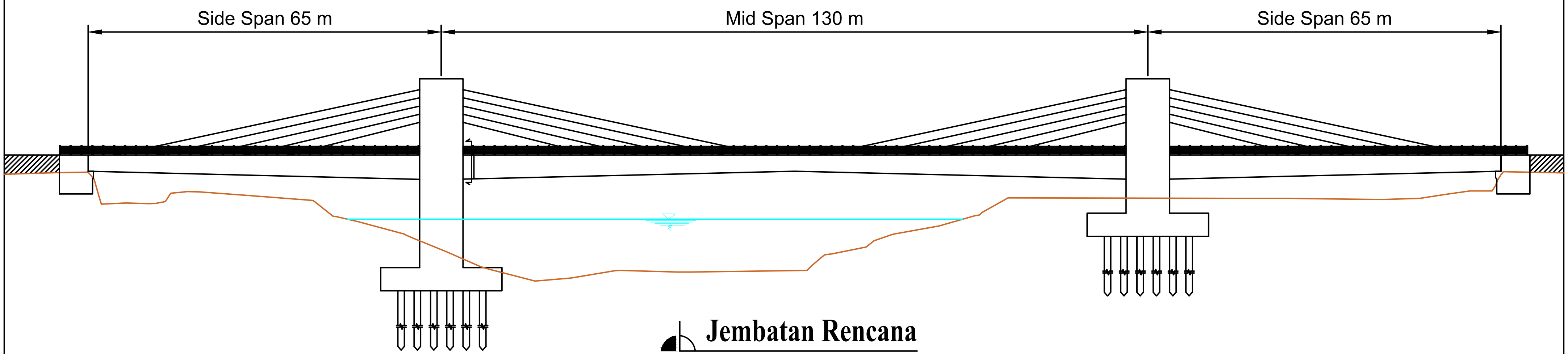
SKALA 1 : 1000



Potongan S0

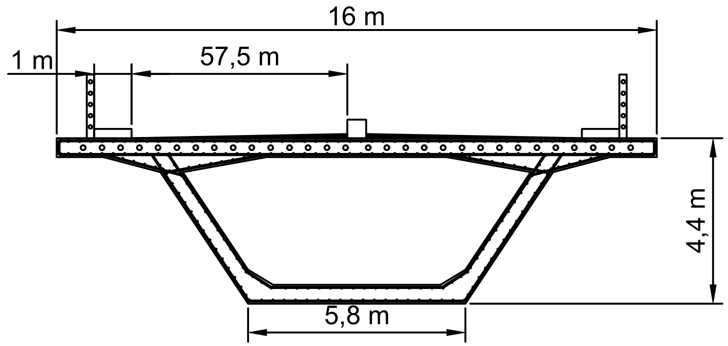
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	25	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

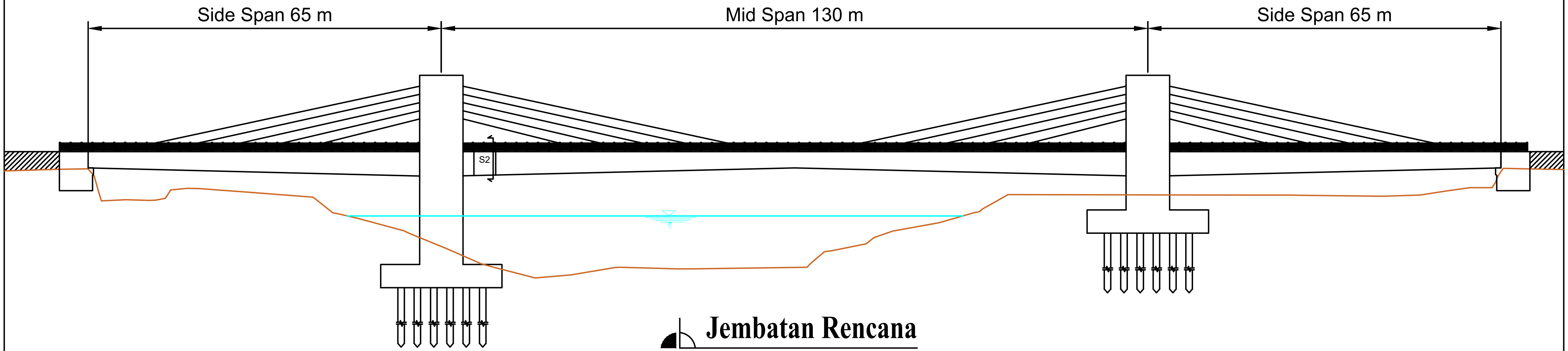
SKALA 1 : 1000



Potongan S1 Mid Span

SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	26	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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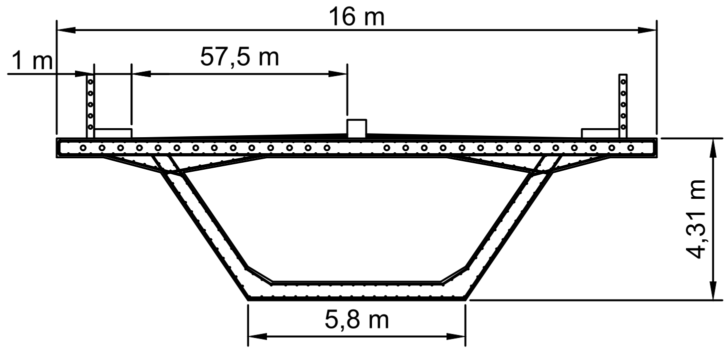
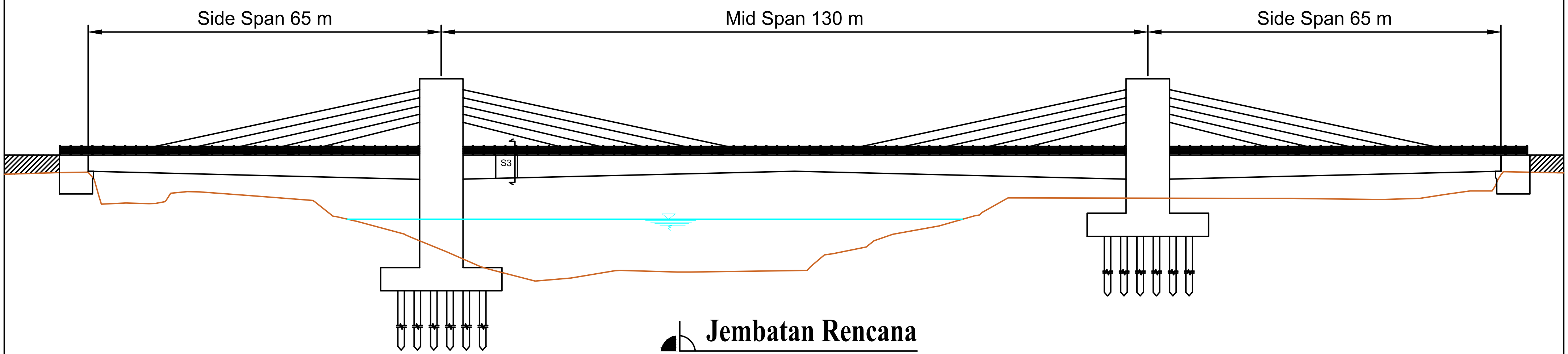


Potongan S2 Mid Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	27	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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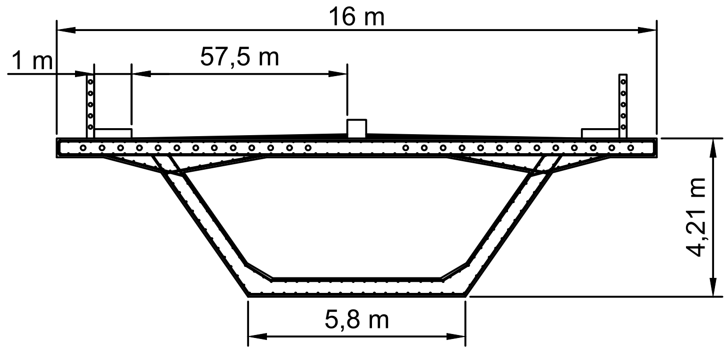
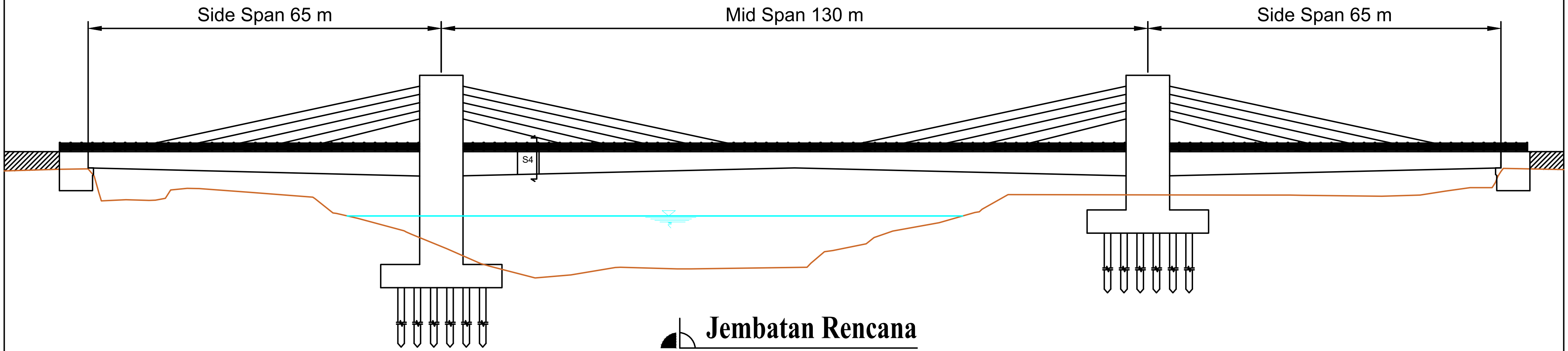


Potongan S3 Mid Span

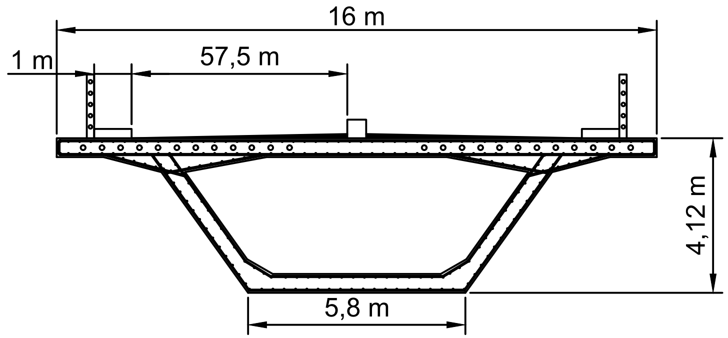
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	28	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

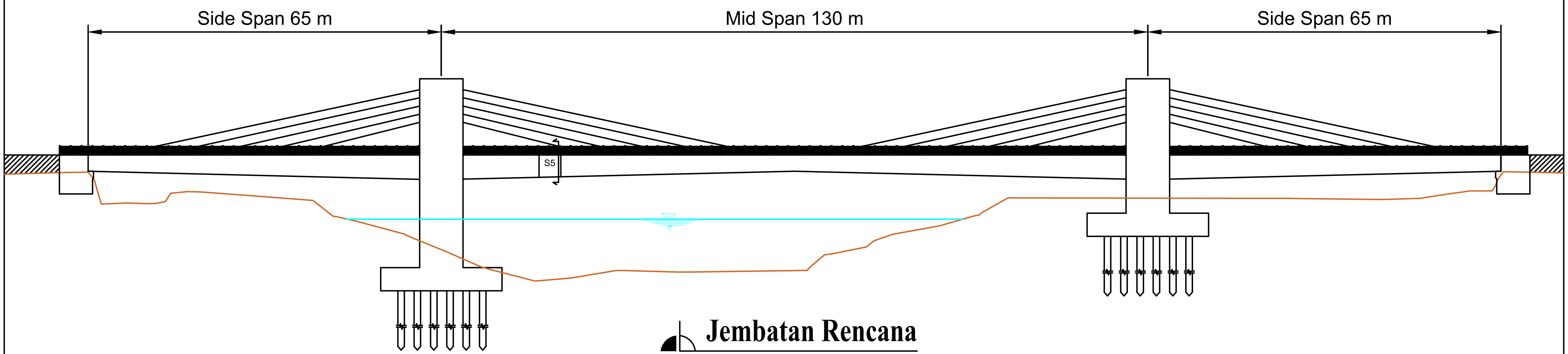
SKALA 1 : 1000



Potongan S4 Mid Span

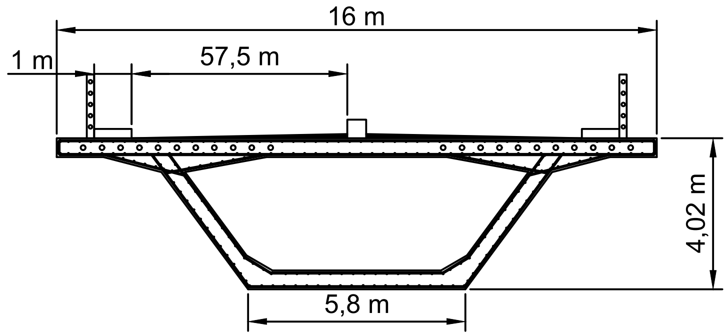
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	29	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

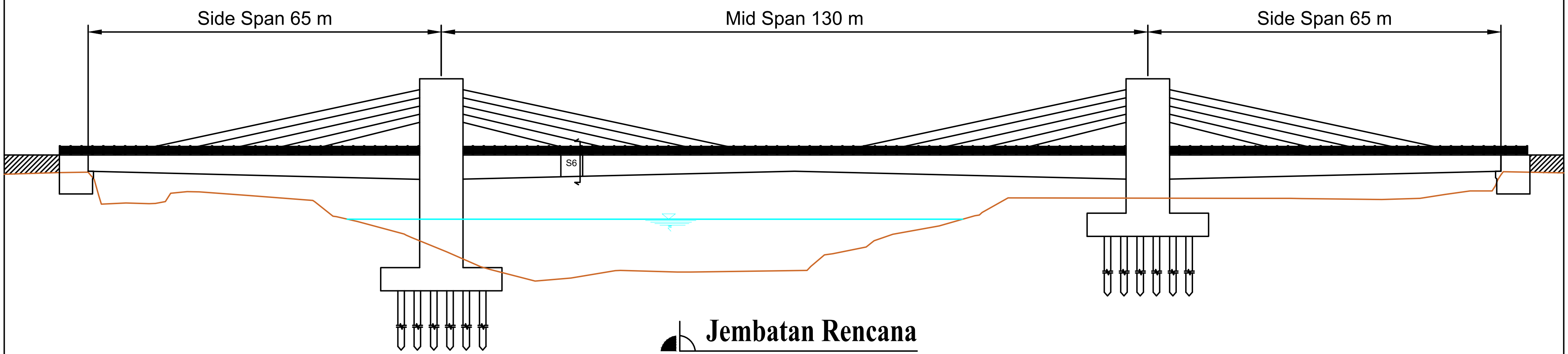
SKALA 1 : 1000



Potongan S5 Mid Span

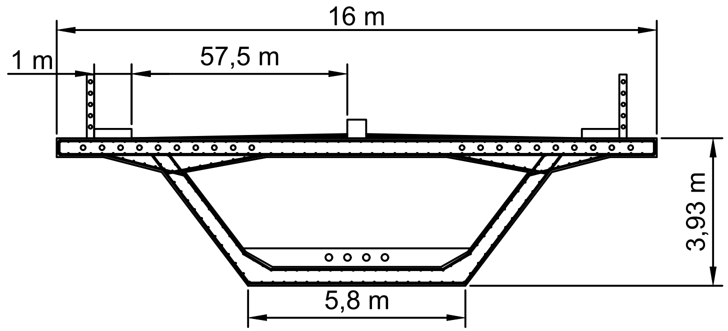
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	30	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

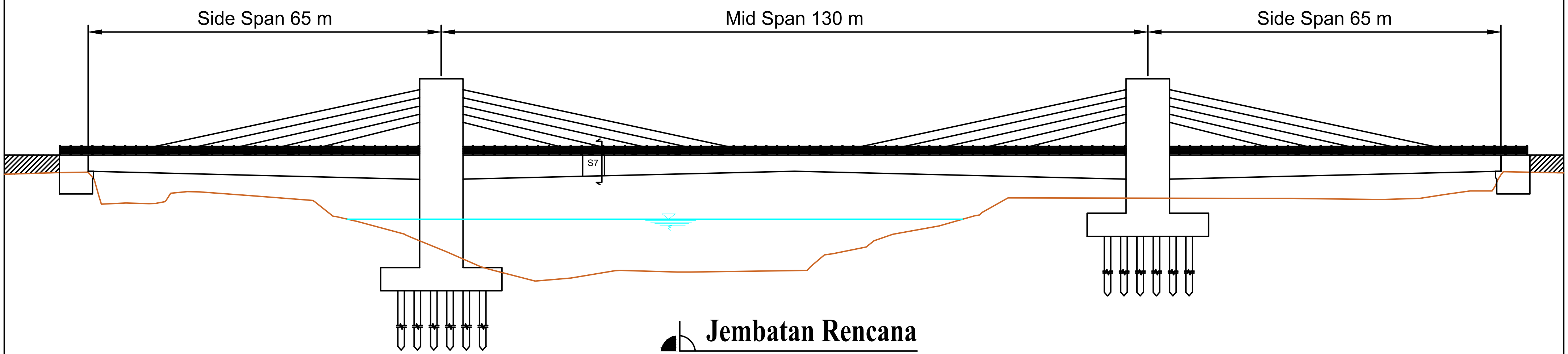
SKALA 1 : 1000



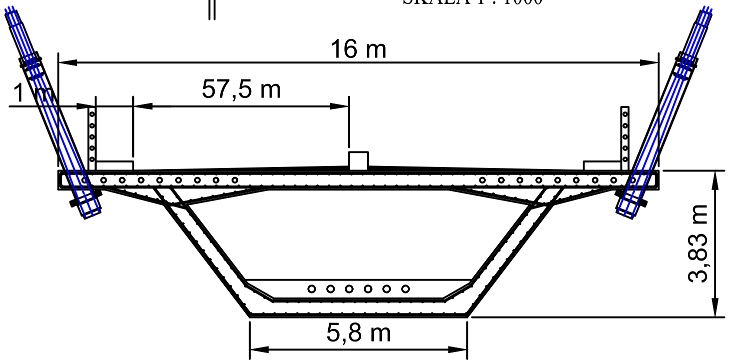
Potongan S6 Mid Span

SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	31	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084

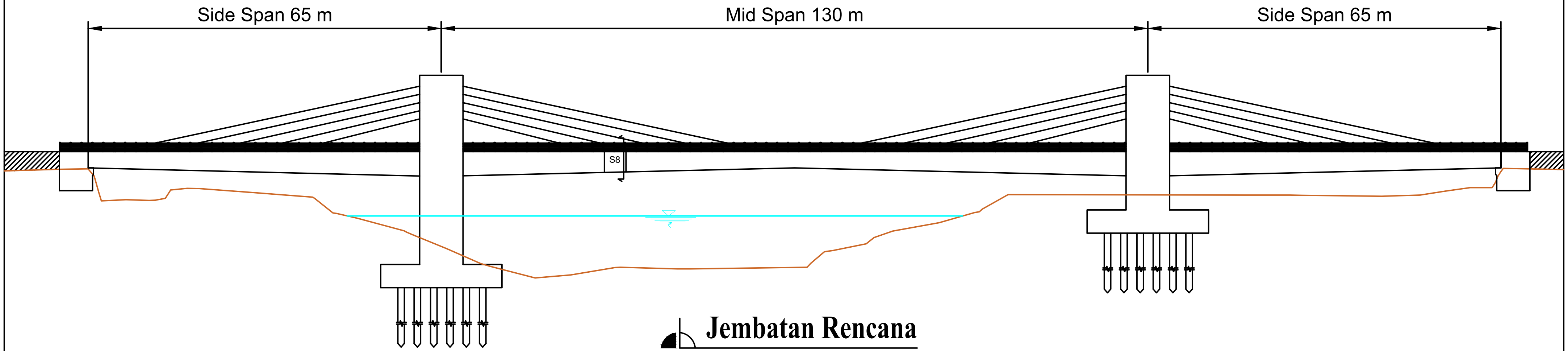


Jembatan Rencana
SKALA 1 : 1000



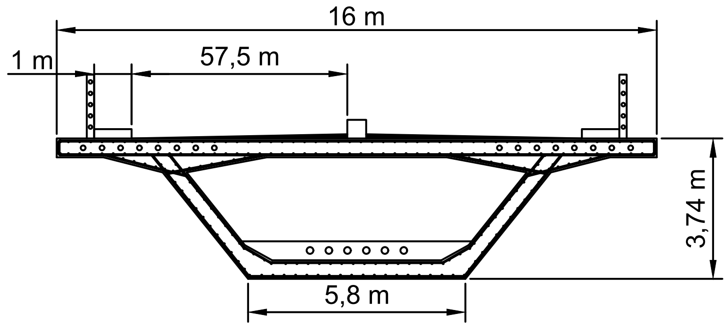
Potongan S7 Mid Span
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	32	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

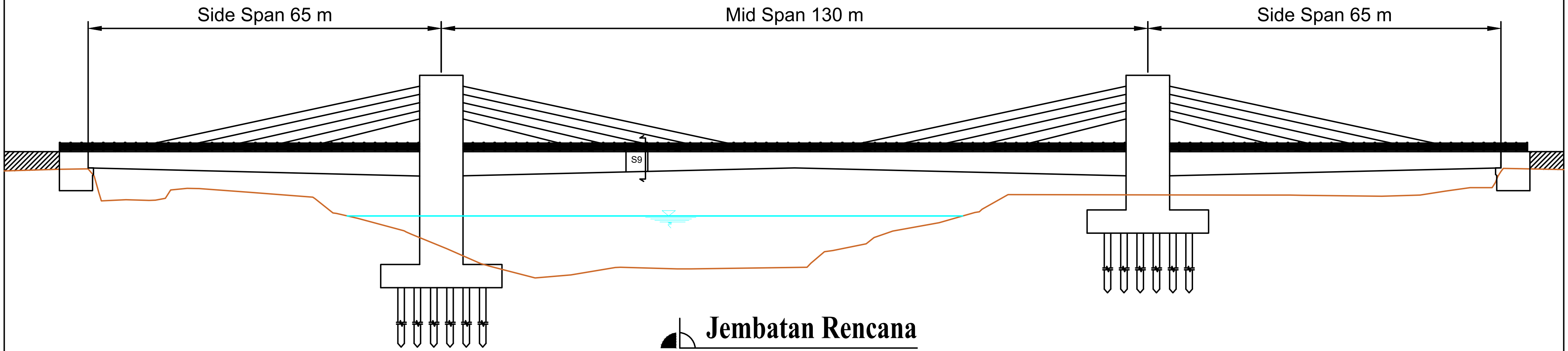
SKALA 1 : 1000



Potongan S8 Mid Span

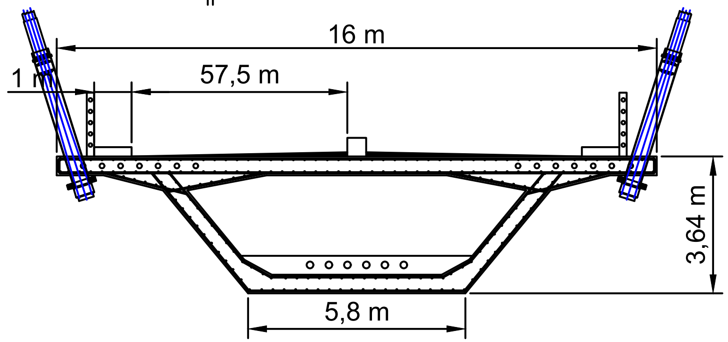
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	33	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

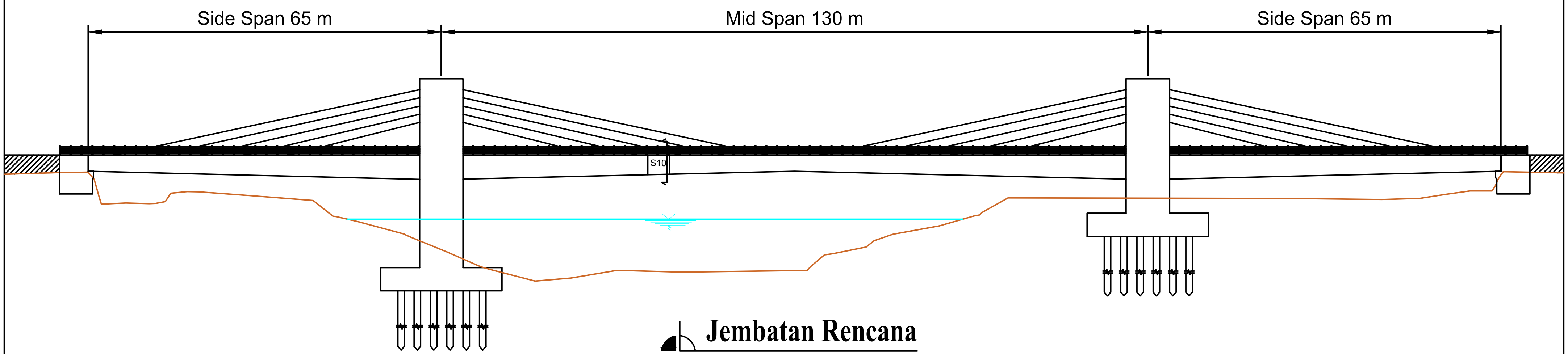
SKALA 1 : 1000



Potongan S9 Mid Span

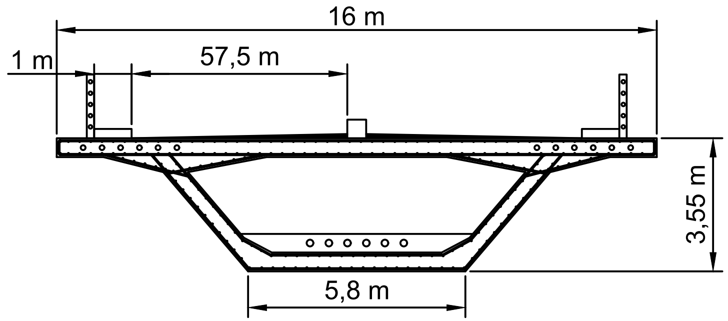
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	34	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

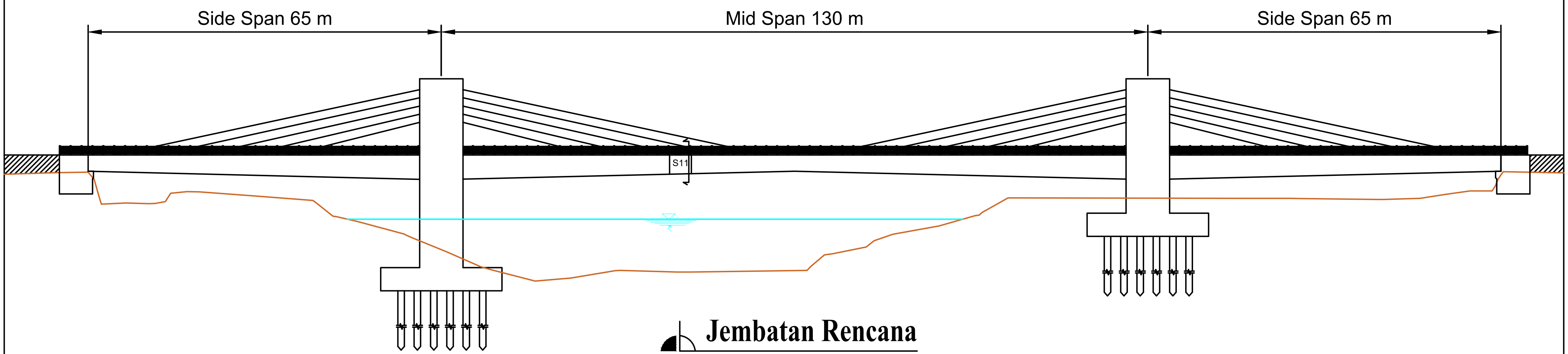
SKALA 1 : 1000



Potongan S10 Mid Span

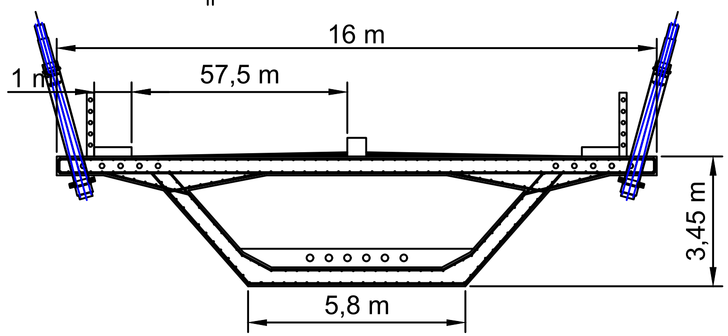
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	35	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

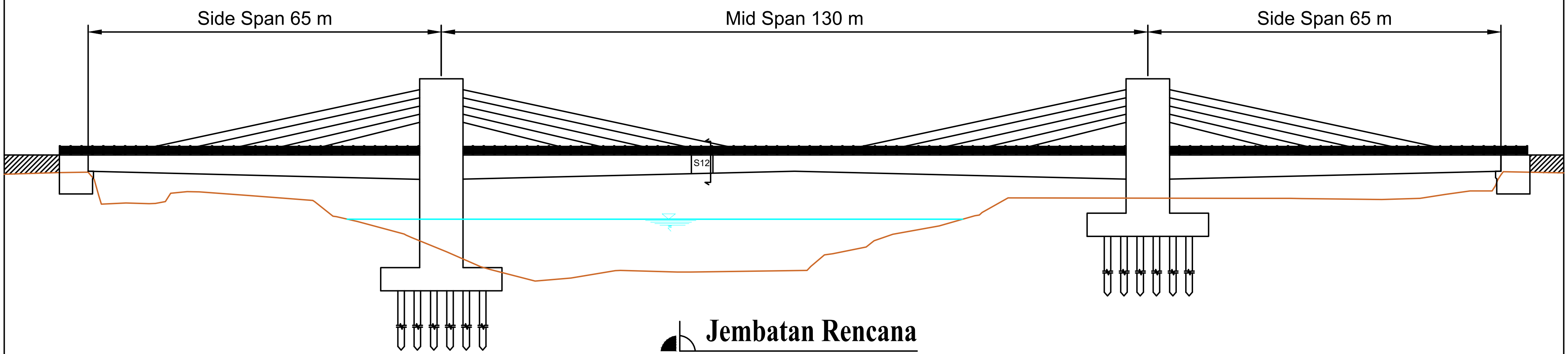
SKALA 1 : 1000



Potongan S11 Mid Span

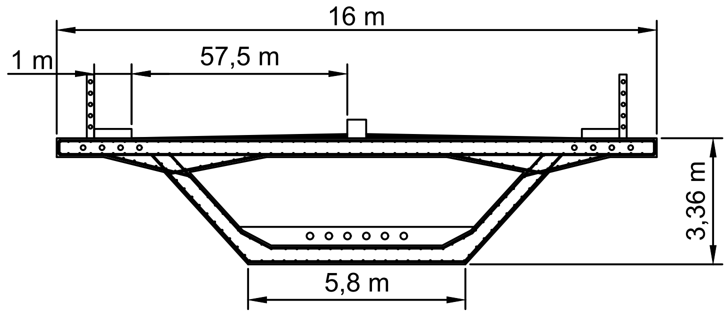
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	36	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

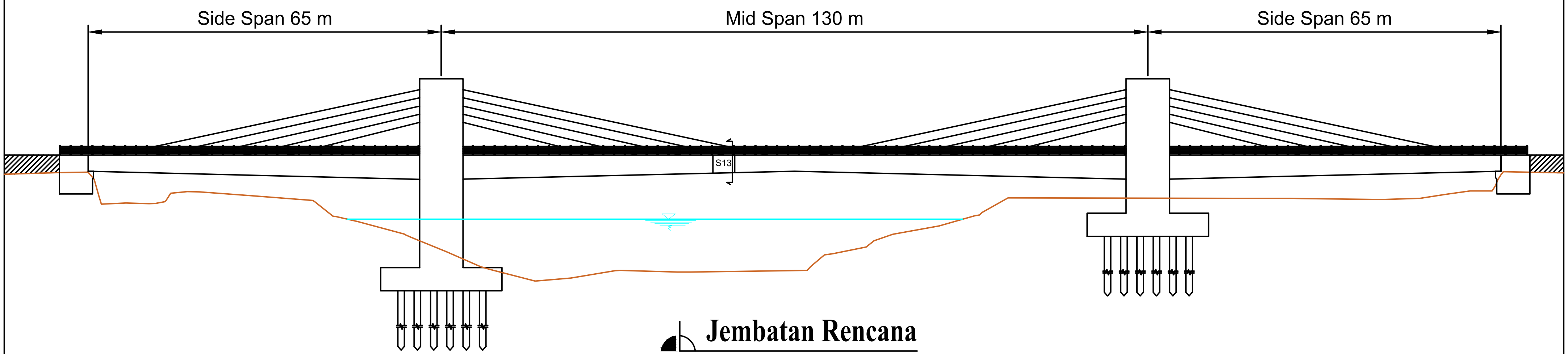
SKALA 1 : 1000



Potongan S12 Mid Span

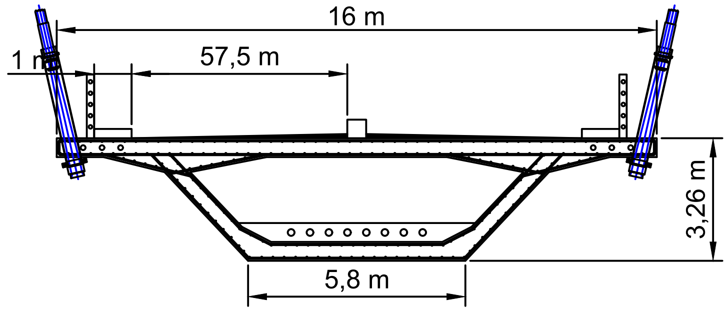
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	37	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

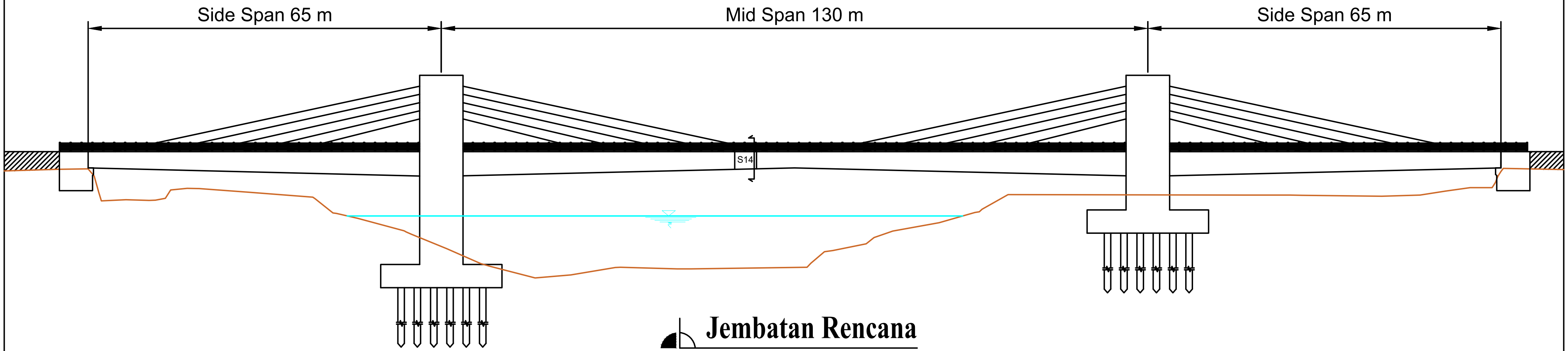
SKALA 1 : 1000



Potongan S13 Mid Span

SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	38	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

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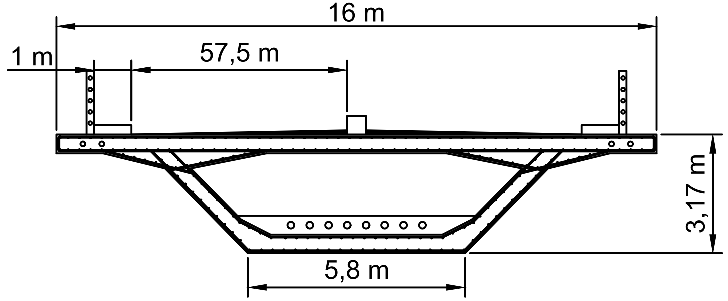
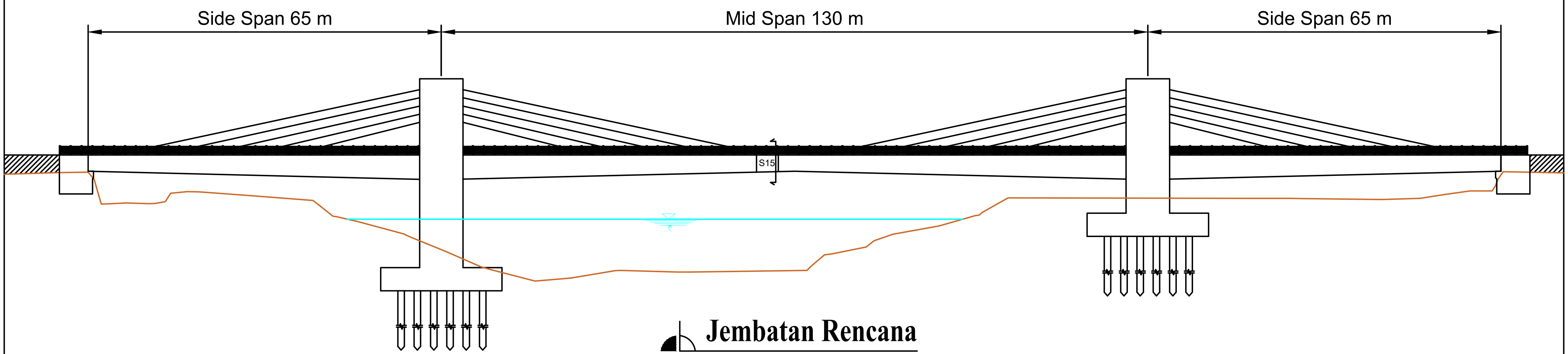


Potongan S14 Mid Span

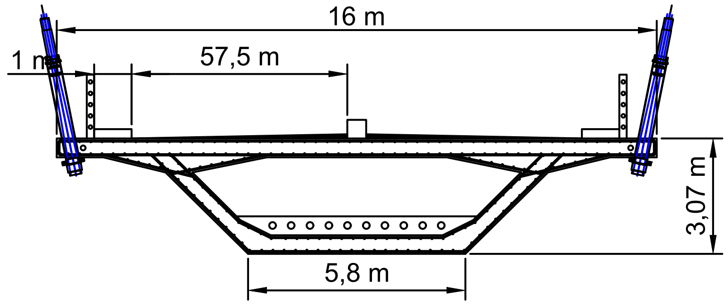
SKALA 1 : 200

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	39	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

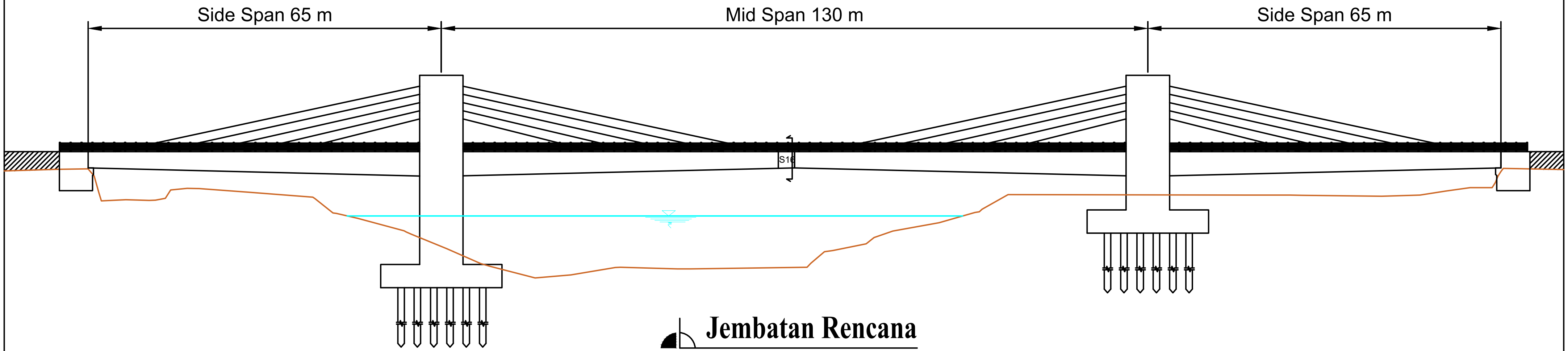
SKALA 1 : 1000



Potongan S15 Mid Span

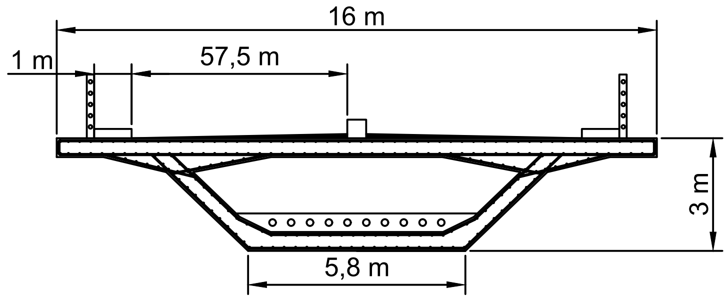
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	40	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Jembatan Rencana

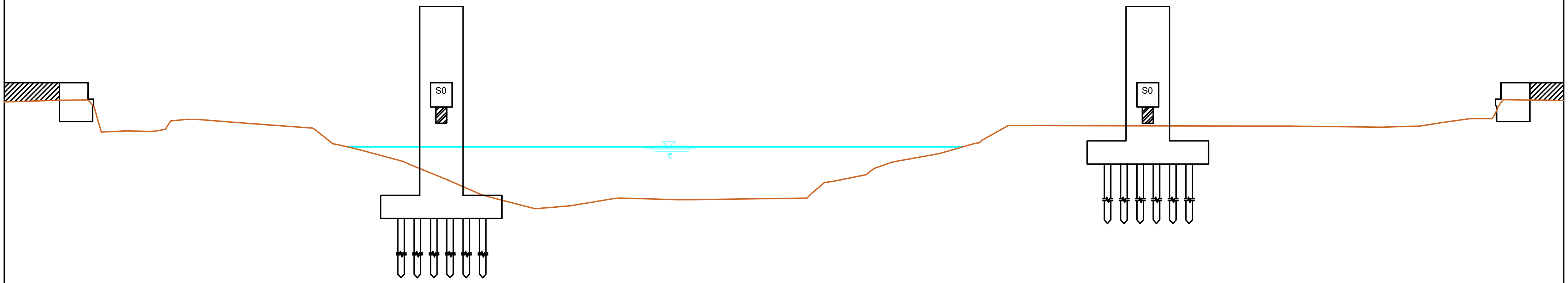
SKALA 1 : 1000



Potongan S16 Mid Span

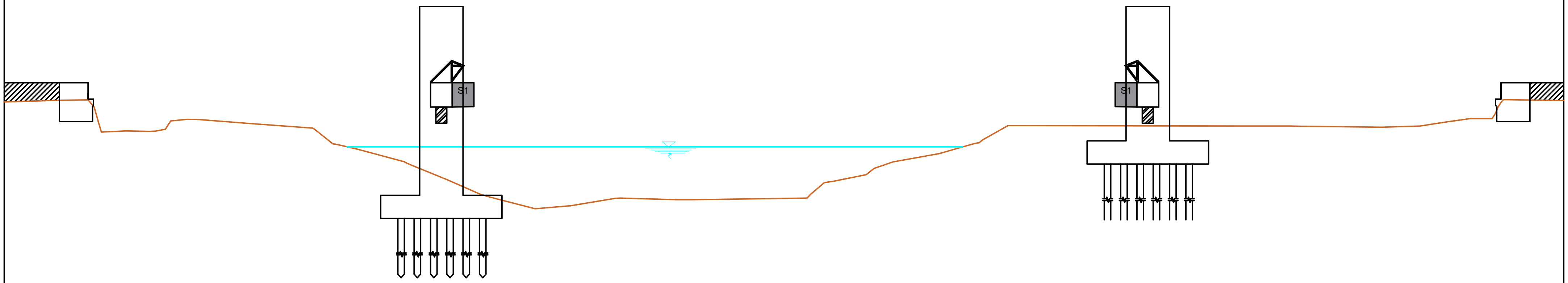
SKALA 1 : 200

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Box Girder	41	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 21
SKALA 1 : 1000

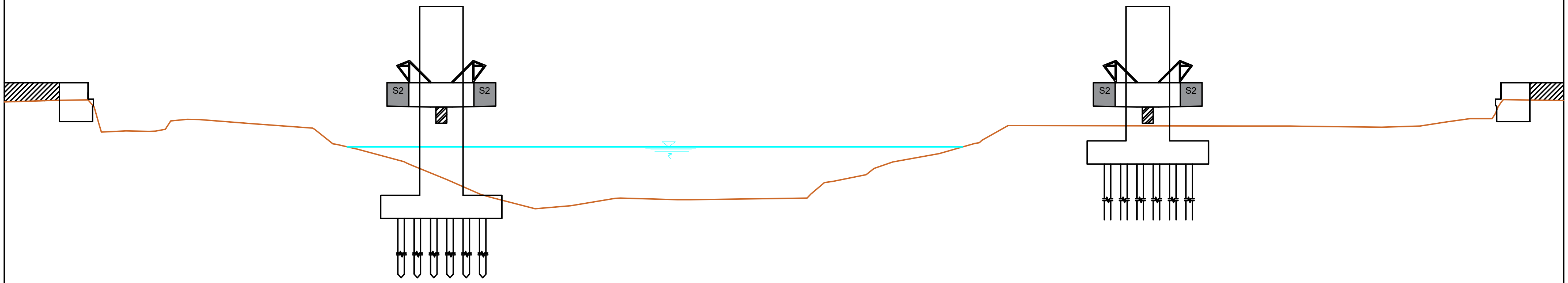
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	42	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 20

SKALA 1 : 1000

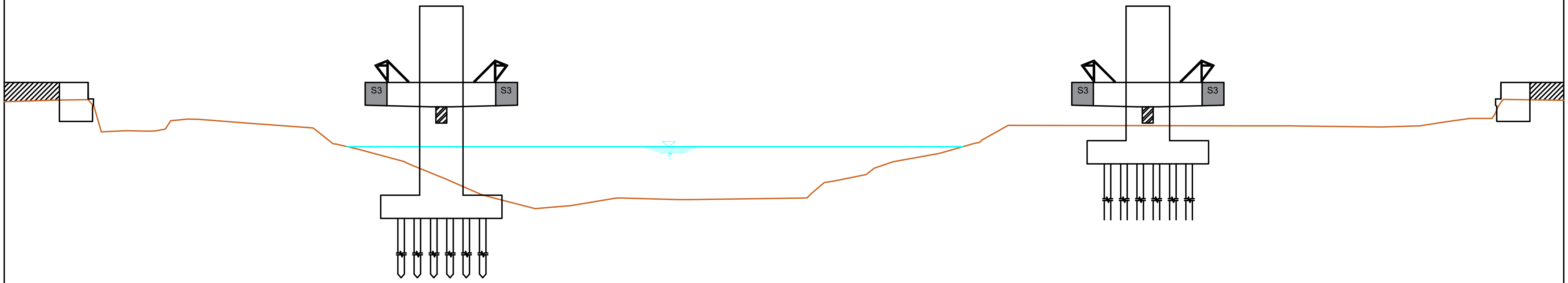
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	43	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 19

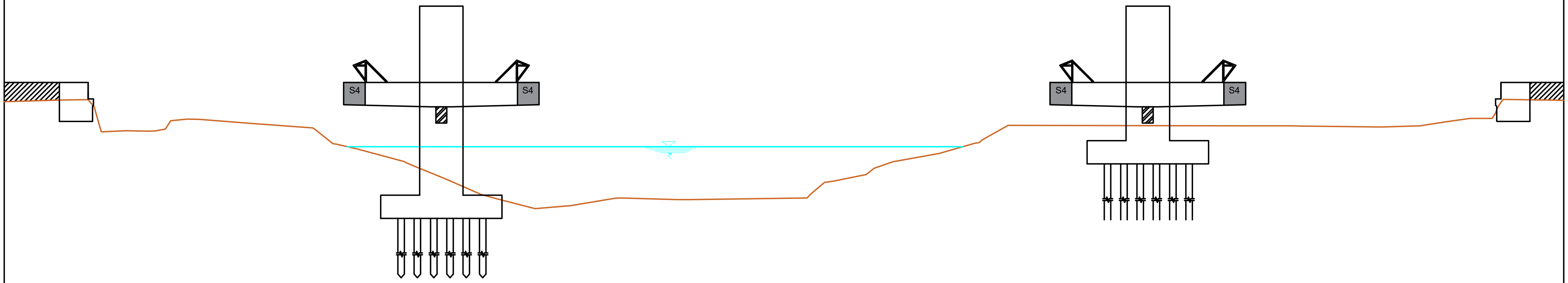
SKALA 1 : 1000

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	44	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 18
SKALA 1 : 1000

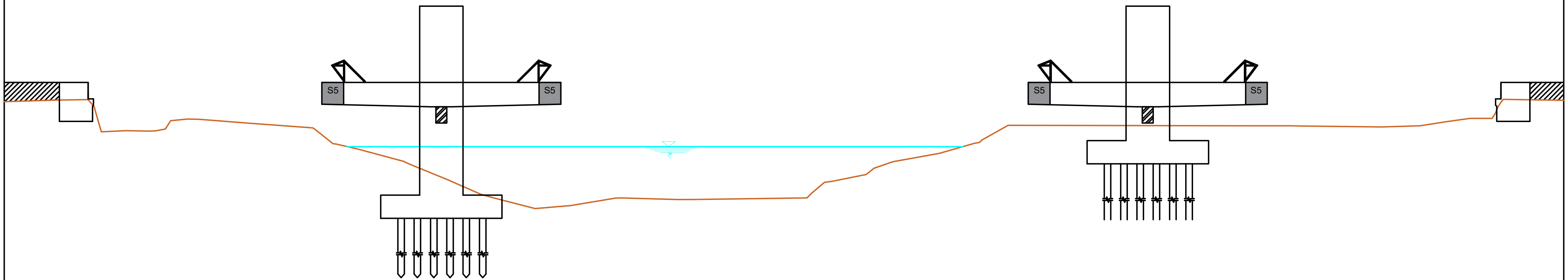
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	**45**	**63**	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 17

SKALA 1 : 1000

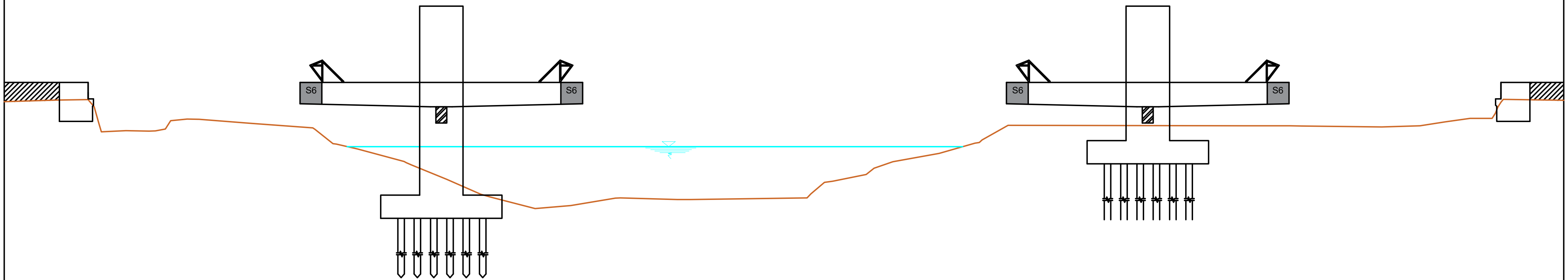
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	46	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 16

SKALA 1 : 1000

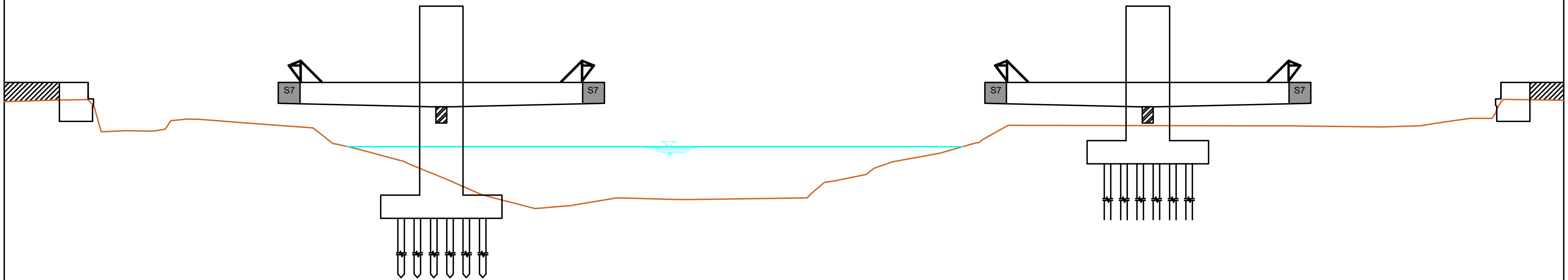
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	47	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 15

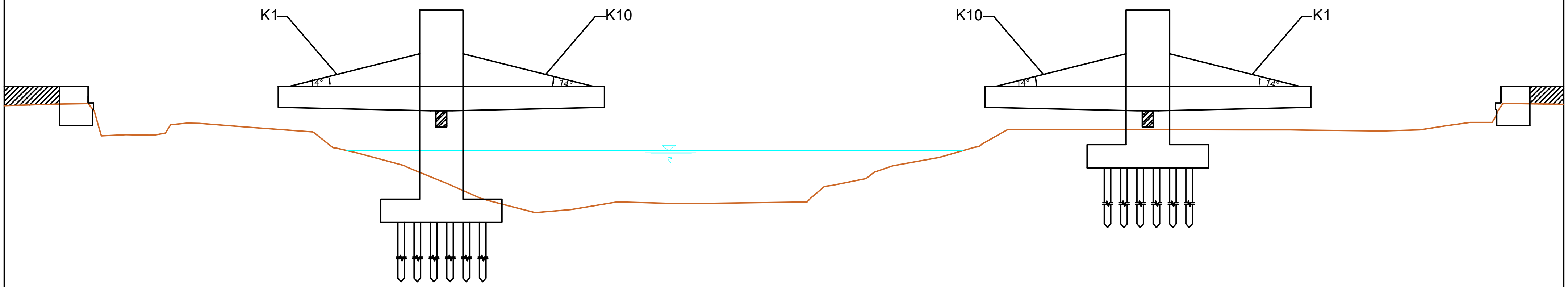
SKALA 1 : 1000

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	48	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 14
SKALA 1 : 1000

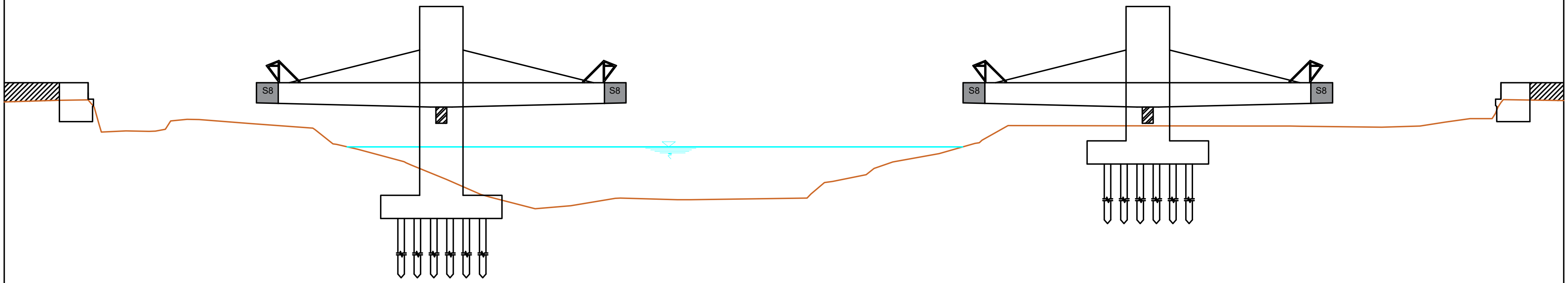
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	49	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 13

SKALA 1 : 1000

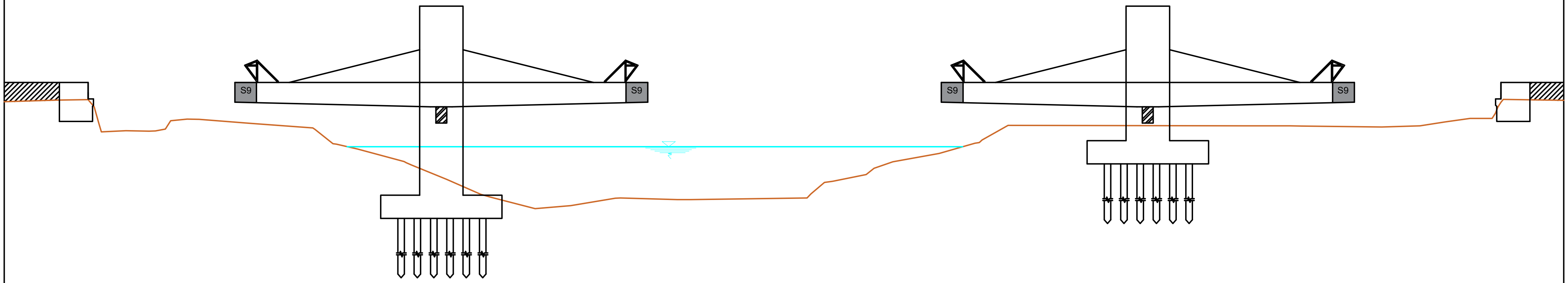
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	50	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 12

SKALA 1 : 1000

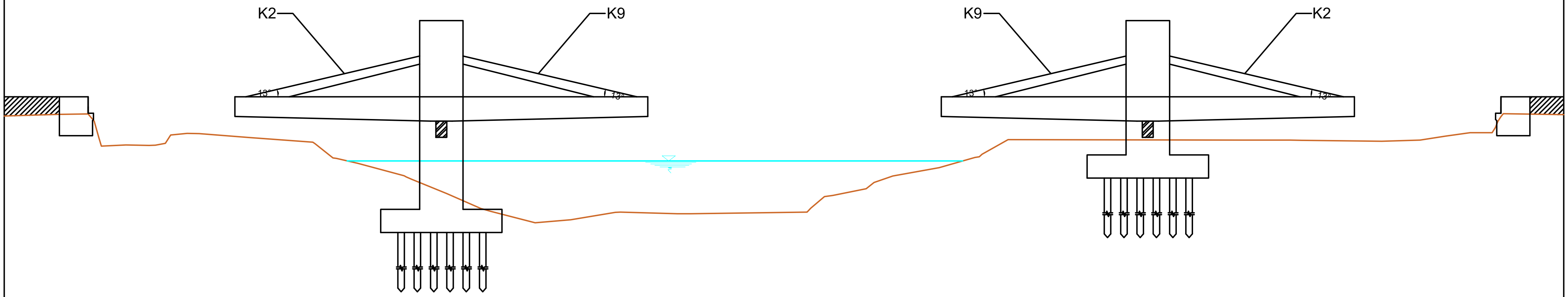
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	51	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 11

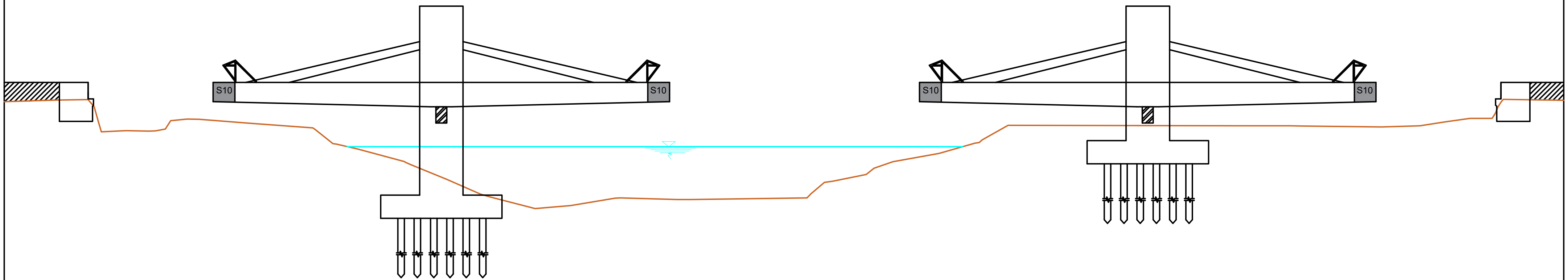
SKALA 1 : 1000

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	52	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 10
SKALA 1 : 1000

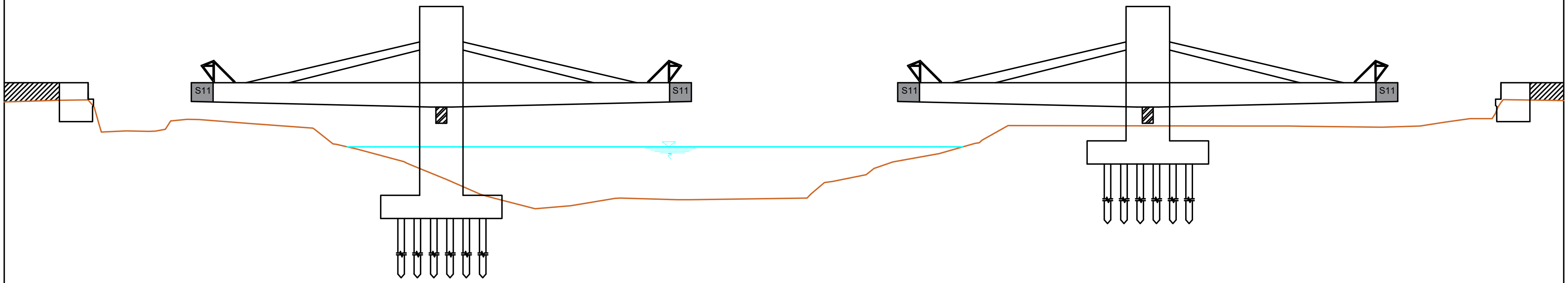
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	53	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 9

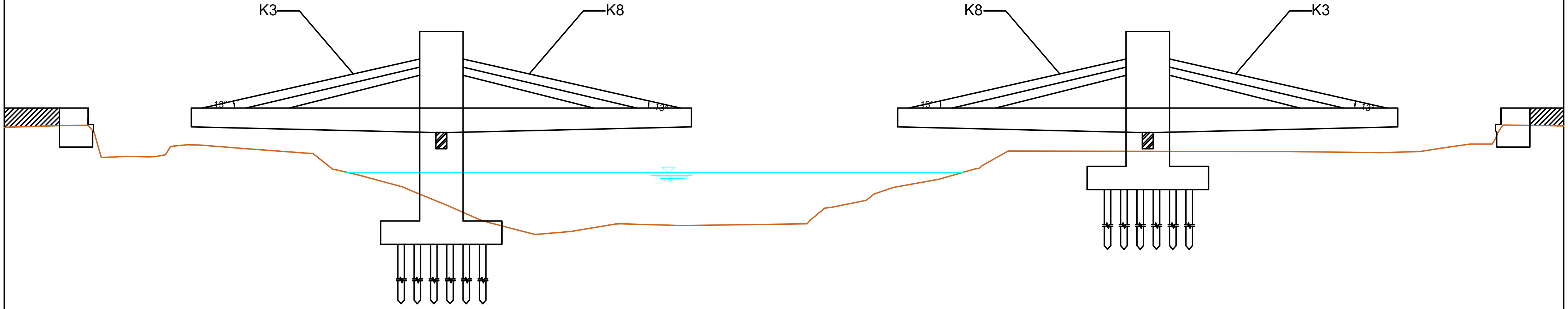
SKALA 1 : 1000

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	54	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 8
SKALA 1 : 1000

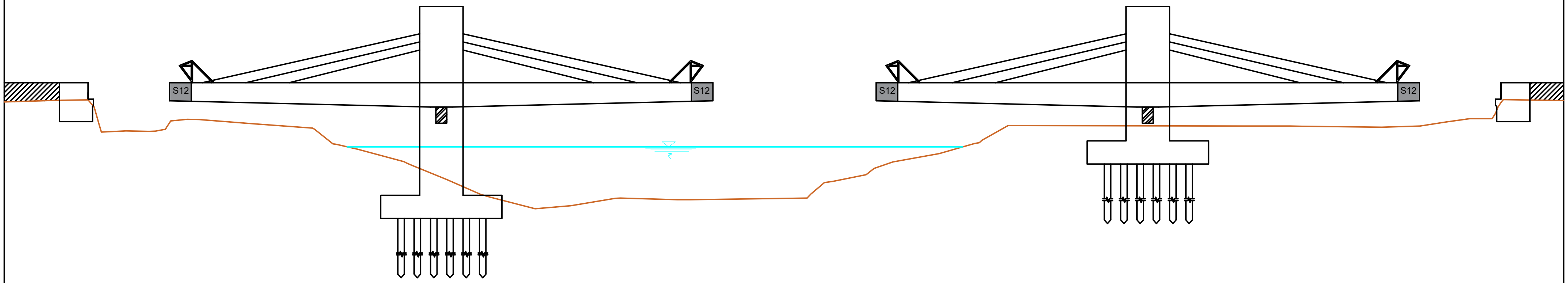
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	55	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 7

SKALA 1 : 1000

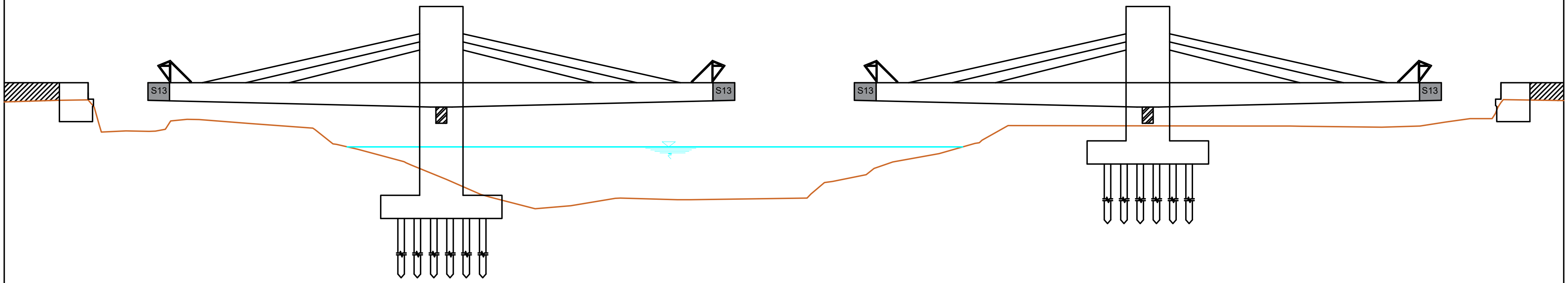
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	**56**	**63**	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 6

SKALA 1 : 1000

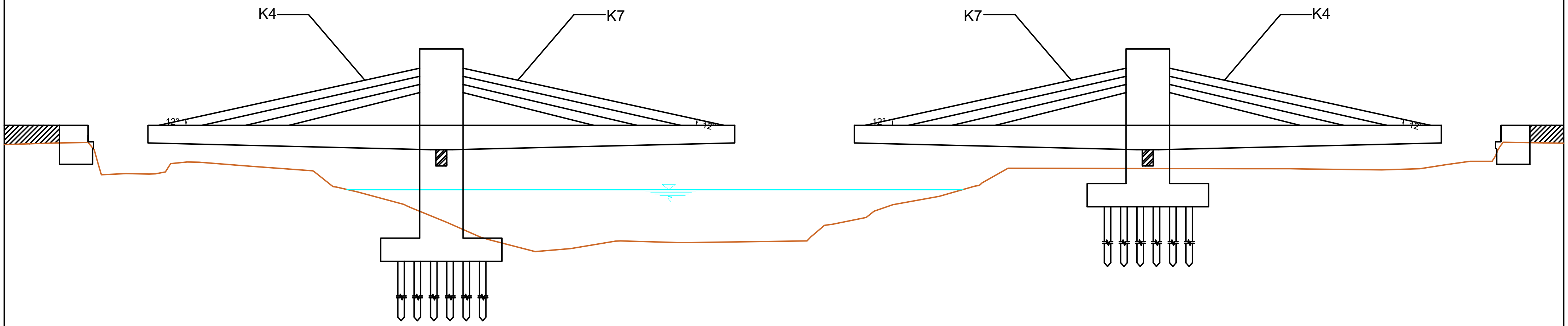
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	57	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 5

SKALA 1 : 1000

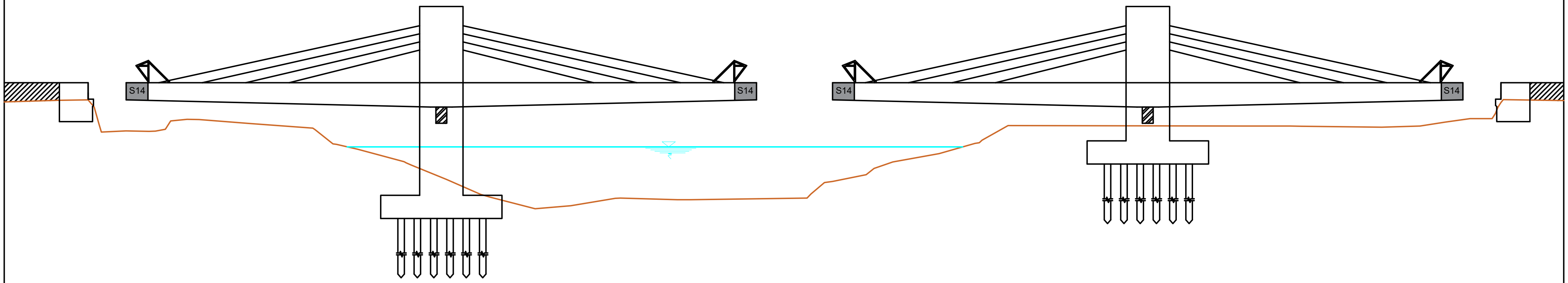
	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	58	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 4

SKALA 1 : 1000

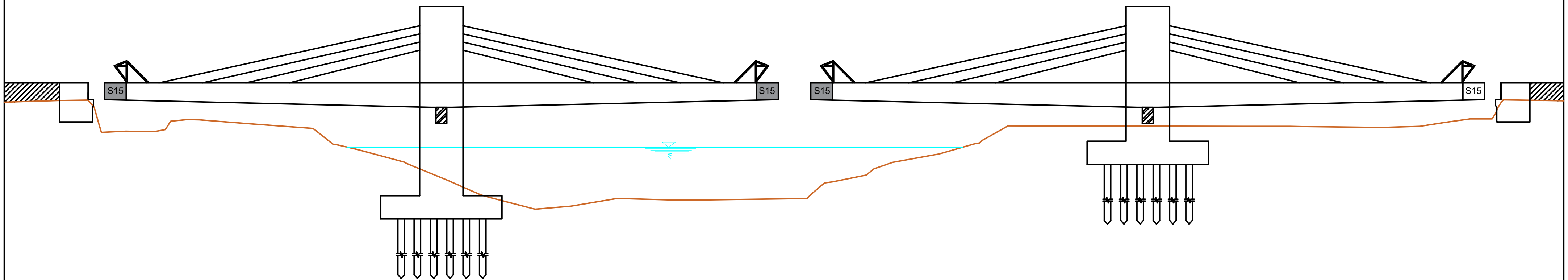
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	59	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 3

SKALA 1 : 1000

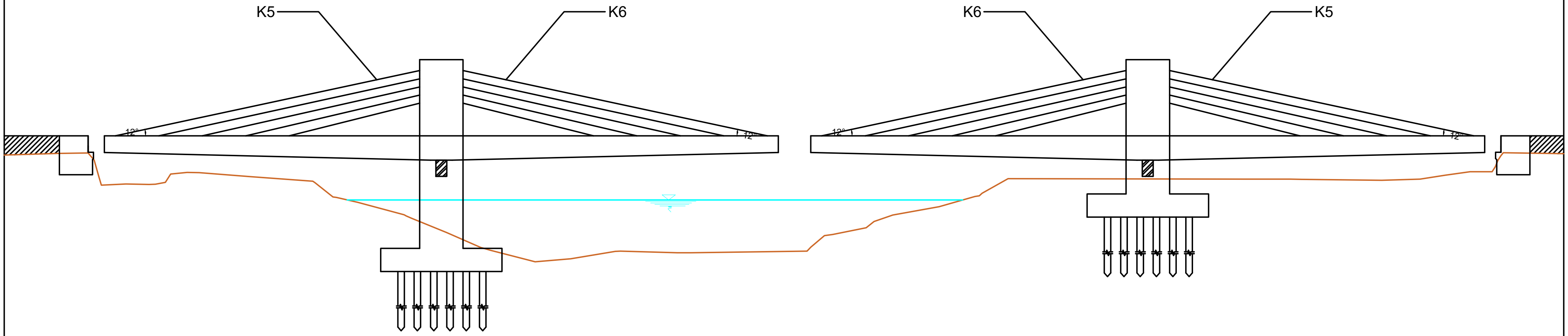
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	60	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 2

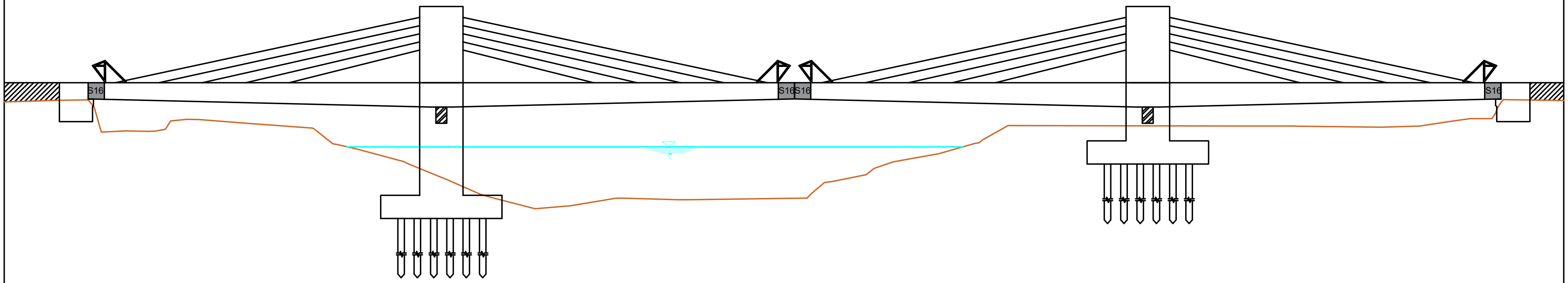
SKALA 1 : 1000

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	**JUDUL TUGAS AKHIR**	**JUDUL GAMBAR**	**NO**	**JUMLAH**	**DOSEN**	**MAHASISWA**
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	**61**	**63**	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 1
SKALA 1 : 1000

	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	62	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084



Stage 0
SKALA 1 : 1000

DEPARTEMEN TEKNIK SIPIL FTSPK - ITS	JUDUL TUGAS AKHIR	JUDUL GAMBAR	NO	JUMLAH	DOSEN	MAHASISWA
	MODIFIKASI PERENCANAAN JEMBATAN CINCIN LAMA WIDANG MENGGUNAKAN SISTEM EXTRADOSED	Staging	63	63	Prof. Dr. Ir. Hidayat Soegiharjo, MS. NIP. 195503251980031004 Data Iranata, ST, MT, Ph.D NIP. 198004302005011002	Muhammad Anhar Praoso NRP. 03111640000084

BIODATA PENULIS

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