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# Review of Superstructure Calculations in Replacement Projects Wai Poka Bridge, Ambon City

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***Abstract :*** *This study is motivated by the aging condition of steel truss bridges in Indonesia, particularly the Wai Poka Bridge in Ambon City, which has caused lane narrowing and reduced structural performance. The study aims to analyze the ultimate moment of the bridge superstructure and evaluate the reinforcement design of conventional concrete girders based on SNI 1725:2016. A descriptive quantitative approach was employed. The population consisted of concrete girder elements within the Maluku national road network, while the sample was purposively selected from the critical dimensions of the Wai Poka Bridge. Research instruments included field surveys and design documents, with data analyzed using SNI-based calculations and structural software. The results indicate that the maximum ultimate moment reached 13,611 kNm and the shear force reached 1,620 kN, with required flexural reinforcement of 33D32 and stirrups of 2D13-200 meeting strength, deflection, and ductility criteria. The conclusion shows that the superstructure replacement design is safe, reliable, and economical, and feasible for similar bridge rehabilitation projects.*

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## Introduction

The development of transportation infrastructure in Indonesia, particularly bridges, is a top priority for the government to improve connectivity between regions separated by natural barriers such as rivers and valleys. Bridges serve to connect disconnected roads, support the smooth flow of vehicle and pedestrian traffic, and contribute significantly to regional economic growth. The bridge superstructure supports traffic and pavement loads, while the substructure and foundations transfer these loads to the ground while maintaining stability against lateral pressure. Witriyatna et al. (2018)

emphasize that the importance of bridges varies among individuals, making them an interesting topic of study in civil engineering.

The Ministry of Public Works and Housing (PUPR) continues to encourage the construction and rehabilitation of national bridges, including in Maluku, to address damage due to age and excessive loads, with a significant budget allocation in 2025. The Wai Poka Bridge in Ambon City, which originally used a steel truss structure with a span of 30 meters and a width of 12 meters, was last rehabilitated in 1993 and is now facing a replacement project by the Ministry of Public Works and Housing to improve the status of the national road network in Region I of Maluku Province. The old structure has narrowed the road, disrupting the smooth flow of transportation, so replacement with a conventional concrete type is planned for economical construction that meets feasibility standards.

The main problem arises from the imbalance between structural strength and construction costs, where bridges must be strong enough to withstand ultimate loads without being excessive to remain economical. Damage to older bridges often causes traffic disruptions, as seen in various rehabilitation cases in Indonesia. For the Wai Poka Bridge, the transition from truss to conventional concrete required an ultimate moment analysis of the superstructure and a review of the reinforcement to ensure its durability against vehicle and environmental loads.

This study reviews the superstructure calculations for the replacement of the Wai Poka Bridge based on the existing design, focusing on the ultimate moment and reinforcement of conventional concrete girders in accordance with the latest SNI. The urgency of this study lies in the need to improve the safety of national infrastructure in eastern Indonesia, where old bridges are prone to failure and hamper accessibility. The novelty of this study is the specific analysis of the transition of truss structures to conventional concrete on a 30-meter span, which is rarely discussed in the current literature, while also providing practical references for the economical design and maintenance of similar bridges.

This study aims to analyze the ultimate moment acting on the superstructure of the Wai Poka Bridge in Ambon City and to review the reinforcement of the superstructure based on the existing replacement design.

## Research Methods

This study uses a descriptive quantitative approach for the numerical structural analysis of the Wai Poka Bridge superstructure, Ambon (30 m span, 12 m width), based on SNI 1725:2016, which effectively tests the ultimate moment and conventional concrete reinforcement through empirical data (Sugiyono, 2021; Creswell & Creswell, 2023). The primary instrument is a field survey in Poka Village, Jl. Y. Syaranamual, and secondary data from PT. Karya Ruata's design and PUPR documents, analyzed descriptively and quantitatively using SNI formulas, structural software, and data triangulation (Sudaryono, 2022; Emzir, 2024). The population includes concrete girder elements replacing steel trusses on the Maluku national network, with purposive sampling on critical dimensions such as effective width, sidewalks, and main reinforcement for analysis efficiency (Sugiyono, 2021; Creswell & Creswell, 2023). The procedure includes field observations, literature studies, and validation of girder reinforcement via structural simulation.

## Results and Discussion

### Backrest Pillar Calculation

The backrest pole measures 25/25, capable of supporting a horizontal load of 100 kg.

1. **Railing Post Load**

| | | | | |
|---|---|---|---|---|
| Distance between railing posts, | L | = | 2 | m |
| Horizontal load on the railing, | H1 | = | 100 | Kg/m |
| Horizontal force on the railing post, | HTP = H1. L | = | 2 | kN |
| Arms against the bottom of the railing post, | Y | = | 0.85 | m |
| Moment on the railing post, | MTP = HTP . Y | = | 1.70 | kN/m |
| Ultimate load factor, | ID card | = | 1.8 | |
| The ultimate moment of the plan, | MU = KTP . MTP | = | 3.06 | kN/m |
| Design ultimate shear force, | VU = KTP . HTP | = | 3.60 | kN/m |


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## 2. Railing Post Reinforcement

### A. Flexible Reinforcement

| | | | | | |
|---|---|---|---|---|---|
| The moment of the ultimate plan, | | Your | = | 3.06 | kNm |
| Concrete quality, | K – 350 | fc' | = | 29.05 | Mpa |
| Steel quality, | U – 28 | fy | = | 280 | Mpa |
| Thickness of railing posts, | | h | = | 250 | mm |
| The distance between the reinforcement and the outside of the concrete, | | d' | = | 30 | mm |
| Modulus of elasticity of steel, | | Ice | = | 2 x 105 | |
| Concrete stress distribution factor, | | $\beta 1$ | = | 0.85 | |

$\rho_b$ = $\beta_1$x 0.85 x fc'/ fy x 600 / ( 600 + fy )
= 0.85 x 0.85 x 29.05 / 280 x 600 / ( 600 + 280 ) = 0.051

$R_{max}$ = 0.75 x$\rho_b$x fy x [ 1 – 1/2 x 0.75 xx fy/ ( 0.85 x fc' )]$\rho_b$
= 0.75 x 0.051 x 280 x [ 1 – ½ x 0.75 x 0.051 x 280 /(0.85 x29.05)]
= 8,387

| | | | | | |
|---|---|---|---|---|---|
| Flexural strength reduction factor, | | $\emptyset$ | = | 0.8 | |
| Shear strength reduction factor, | | $\emptyset$ | = | 0.6 | |
| The moment of the ultimate plan, | | Your | = | 3.06 | kNm |
| Width of railing post, | | b | = | 250 | mm |
| Effective thickness of railing post, | | d = h – d' | = | 220 | mm |
| Nominal moment of plan, | | Mn = Mu /$\emptyset$ | = | 3.82 | kNm |

Moment resistance factor, $R_n$ = $M_n$x 10-6/( b x d2 )
= 3.82 x10-6/(250 x 2202) = 0.32

$R_n$ < $R_{max}$ ( OK )

Required Reinforcement Ratio,

ρ = 0.85 x fc'/ fy x [ 1 -$\sqrt{[1-2\,x\;\;Rn/\,(0.85\,x\,fc')}$

= 0.85 x 29.05 / 280 x [1- $\sqrt{[1-2\,x\,0{,}32/(0.85\,x29{,}05)}$ = 0.0012

Minimum reinforcement ratio, $\rho_{min}$= 0.5 / fy = 0.002

The ratio used, $\rho_{min}$ = 0.002

The required reinforcement area, $A_S$= ρ xbxd
= 0.002 X 250 X 220 = 110 mm

Diameter of reinforcement used, D = 12 mm

The amount of reinforcement required, n = As / ( / 4 x D2)$\pi$
-= 110 /(3.14/4x122) = 0.97

Used 2D 12 reinforcement

### B. Shear Reinforcement

Design ultimate shear force, Vu = 3.60 = 3600 N

Concrete shear strength, $Vc = \frac{\sqrt{fc'}}{6}\,x\,b\,x\,d\,x\,10^{-3}$

$= \frac{\sqrt{29{,}05}}{6}\,x\,250\,x\,220\,x10^{-3}$ = 49,389 N

$\emptyset\,x\,Vc$ = 0.6 x 49.389 = 29,633 N

$Vs = \frac{Vu-(\Phi\,x\,Vc)}{\Phi}$

$= \frac{3600-(0{,}6\,x\,29{,}633)}{0{,}6}$ = 5970,367 N

For structural stability, minimum reinforcement is required.

Smak = 0.5 xd
= 0.5 x 220 = 110 Mm

A spacing of 110 mm is used with a minimum reinforcement area,

$A_v$ = 1/3 x$\sqrt{fc'}\,x\,b\,x\,S\,/fy$
= (1/3 x$\sqrt{29{,}05}\,x\,250\,x\,110)\,/280$ = 176.45 mm

Stirrups with a cross section of 2 Ø 10 mm are used

The area of the shear reinforcement of the stirrup,

$A_v = \frac{\pi}{4}\,x\,\emptyset^2\,x\,2$

$= \frac{3{,}14}{4}\,x\,10^2\,x\,2$ = 157 mm

The required spacing of the stirrup shear reinforcement,

S = $Av\,.\,fy\,.\,\frac{d}{Vs}$

$= 157\,.\,280\,.\,\frac{220}{5970{,}367}$ = 1619.87 mm

So the shear reinforcement used is 2$\emptyset$10 – 15

## Sidewalk Calculation

## 1. Bridge Data

| | | | | |
|---|---|---|---|---|
| Thickness of floor slab, | h | = | 20 | Cm |
| Thick asphalt, | t | = | 5 | Cm |
| Thickness of the rainwater layer, | th | = | 5 | Cm |


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| | | | | | |
|---|---|---|---|---|---|
| Thickness of concrete cover, | | p | = | 3 | Cm |
| The loads reviewed throughout, | | L | = | 2 | M |
| Concrete quality, | K = 300 | fc' | = | 24.90 | Mpa |
| Steel quality, | U = 28 | fy | = | 280 | Mpa |
| Specific gravity of concrete, | | toilet | = | 2400 | Kg/m3 |
| Asphalt specific gravity, | | Wow | = | 2200 | Kg/m3 |
| Specific gravity of rainwater, | | Ww | = | 1000 | Kg/m3 |
| The load that occurs according to PPPJJR 87 chapter III article 1 (2) 2.5 which works: | | | | | |
| Horizontal, | | H1 | = | 100 | Kg/m1 |
| Kerb, | | H2 | = | 500 | Kg/m1 |
| Sidewalk, | | H3 | = | 500 | Kg/m2 |

## 2. Dead load on pavement

**Table 1. Dead load on pavement**

| No | Type | Thick (m) | Wide (m) | Long (m) | Heavy (kg/m) | Burden (kg/m) |
|---|---|---|---|---|---|---|
| 1 | Self weight of the floor | 0.20 | 1.25 | 1 | 2400 | 600 |
| 2 | Curb and curb weight | 0.25 | 1 | 1 | 2400 | 600 |
| 3 | Weight of rainwater | 0.05 | 1 | 1000 | | 50 |
| | | | | | Σq1 | 1250 |

Source: Calculation results

Moment Mq1 =½ . q1 . L2
=½ . 1250 . 12
= 625 kg/m

## 3. Centralized loading

**Table 2. Concentrated loads**

| Type | Thick (m) | Wide (m) | Tall (m) | Heavy (kg/m) | Burden (kg/m) |
|---|---|---|---|---|---|
| Self weight of the backrest | 0.25 | 0.25 | 1.25 | 2400 | 187.5 |
| Horizontal load | | | | 100 | 100 |
| | | | | Σp1 | 287.5 |

Source: Calculation results

Moment Mp1 = p1. L
= 287,5 . 1 = 287.5 kg/m

## 4. Live load

Sidewalk load q2 = 500 kg/m

Moment Mq2 =½ . q2 . L2
=½ . 500 . 1
= 250 kg/m

## 5. Due to horizontal curb loads

Curb load P2 = 500 kg/m
h = 0.25 m

Moment Mp2 = p2. h
=500 . 0.25
= 125 kg/m

## 6. Total moment that occurred

M total = Mq1 + Mp1 + Mq2 + Mp2
= 625 + 287.5 + 250 + 125 = 1287.5 kg/m

## 7. Looking for latitude

D =( q1 + q2 ) . L . p1
= (1250 + 500) . 1. 287.5
= 503125 kg/m

## Floor Plate Calculation

## 1. Superstructure Data

| | | | | |
|---|---|---|---|---|
| Types of bridges | | : | Reinforced concrete | |
| Thickness of bridge floor slab | `ts | = | 0.20 | m |
| Thickness of asphalt layer + overlay | ta | = | 0.1 | m |
| Thickness of puddles of rainwater | th | = | 0.05 | m |
| Number of girders | n | = | 5 | Fruit |


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| | | | | |
|---|---|---|---|---|
| Distance between girders | s | = | 2 | m |
| Traffic lane width | b1 | = | 9 | m |
| Sidewalk width | b2 | = | 1.00 | m |
| Total width of the bridge | B | = | 11 | m |
| Bridge span length | L | = | 30.00 | m |

## 2. Structural Materials

| | | | | |
|---|---|---|---|---|
| Concrete quality: | | | | |
| Concrete compressive strength, | fc' | = | 30 | Mpa |
| Elastic modulus, | Ec = 4700 $\times \sqrt{fc'}$ | = | 25742.96 | MPa |
| Poison number, | $\upsilon$ | = | 0.20 | |
| Coefficient of linear expansion for concrete: | $\alpha$ | = | 1 10-5× | /℃ |
| Steel quality: | | | | |
| For reinforced steel with Ø>12 mm: | U | = | 42 | |
| Yield stress of steel | Fy = U 10× | = | 420 | Mpa |
| For reinforcing steel with Ø<12 mm: | U | = | 28 | |
| Yield stress of steel | Fy = U 10× | = | 280 | Mpa |
| Specific Gravity: | | | | |
| Weight of reinforced concrete | toilet | = | 25.00 | kN/m3 |
| Weight of unreinforced concrete (rebate concrete) | w'c | = | 24.00 | kN/m3 |
| Solid asphalt weight | Wow | = | 22.00 | kN/m3 |
| Specific gravity of water | Ww | = | 9.80 | kN/m3 |

## 3. Bridge Floor Plate Load Analysis

### A. Self Weight (MS)

| | | | | | |
|---|---|---|---|---|---|
| Ultimate load factor | | KMS | = | 1.30 | |
| Viewed wide bridge floor slab | | B | = | 1.00 | m |
| Thickness of bridge floor slab | | h = ts | = | 0.20 | m |
| Weight of reinforced concrete | | toilet | = | 25.00 | kN/m3 |
| Self weight | QMS = b* h * wc | QMS | = | 5.00 | kN/m3 |

### B. Additional Dead Load (MA)

Ultimate load factor: KMA = 2.00

**Table 3. Additional dead load of floor slab**

| NO | TYPE | THICK (m) | HEAVY (kN/m3) | BURDEN kN/m3 |
|---|---|---|---|---|
| 1 | Asphalt layer + overlay | 0.1 | 22.00 | 2.20 |
| 2 | Rainwater | 0.05 | 9.80 | 0.49 |
| | Additional dead load: | | QMA = | 2.69 |

Source: Calculation results, 2024

### C. Truck Load "T" (TT)

| | | | | |
|---|---|---|---|---|
| Ultimate load factor: | Summit | = | 1.80 | |
| The live load on the bridge floor is in the form of double wheels by a truck (load T) with a size of: | T | = | 112.50 | kN |
| Dynamic load factor for truck loading is taken as: | DLA | = | 30 | % |
| Truck load "T" : | PTT = (1 + DLA)* T | = | 146.25 | kN |

### D. Wind Load (EW)

Ultimate load factor: KEW = 1.20

The additional horizontal uniform line load on the bridge floor surface due to wind blowing vehicles over the bridge is calculated using the formula:

TEW = 0.0012*Cw*(Vw)2 kN/m

with,

| | | | | | |
|---|---|---|---|---|---|
| Cw | : | Drag coefficient | = | 1.20 | |
| VW | : | Wind velocity | = | 35.00 | m/sec |
| TEW | = | 0.0012*Cw*(Vw)2 | = | 1,764 | kN/m |

The wind-blown vertical midwife is a side midwife of the vehicle with height above the bridge floor h = 2.00 m

Distance between vehicle wheels x = 1.75 m

Wind Load (TEW)

Service (S) : Ultimate (U) :


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| | | | | | | |
|---|---|---|---|---|---|---|
| VIEW | = | 30 m/sec | | VIEW | = | 35 m/sec |
| TEW | = | 0.0012 x 1.2 x (30 m/sec)2 | | TEW | = | 0.0012 x 1.2 x (35 m/sec)2 |
| | | 1,296 kN/m2 | | | | 1,764 kN/m2 |
| PEW | = | 1/2*h/x*TEW | | PEW | = | 1/2*h/x*TEW |
| | | 0.741 Kn | | | | 1,008 kN |

E. Effect of Temperature (ET)

Ultimate load factor: KET = 1.20

To calculate the stress and deformation of the structure that occurs due to the influence of temperature, a temperature difference of half the difference between the maximum temperature and the average minimum temperature on the bridge floor is taken.

| | | | | | |
|---|---|---|---|---|---|
| Average maximum temperature: | | Tmax | = | 40.00 | °C |
| Average minimum temperature: | | Tmin | = | 15.00 | °C |
| Temperature difference on the slab: | T = (Tmax-Tmin)/2 | ΔT | = | 12.50 | °C |
| Coefficient of linear expansion for concrete: | | $\alpha$ | = | 1 10-5× | ℃ |
| Modulus of elasticity of concrete: | | Ec | = | 25,742,960 | kPa |

## 4. Moments on the Bridge Floor

The maximum moment on the floor slab is calculated based on the one way slab method with the following loads:

Self weight QMS = 5.00kN/m
Dead load QMA = 2.69kN/m
Truck load PTT = 146.25kN
Wind load PEW = 1.01kN
Temperature T = 12.50 °C

The field moment and support moment coefficients for continuous spans with uniform, concentrated, and temperature loads are as follows:

k = moment coefficient
girder spacing S = 2 m
For uniform load Q: M = k * Q * s2
For concentrated load P : M = k * P * s
For temperature load, ΔT : M = k *$\alpha$* DT *Ec *s3

A. Moment due to self-weight (MS):

| | | | | | | |
|---|---|---|---|---|---|---|
| Moment of support, | MMST | = | 1/12 * QMS * s2 | = | 1.67 | kNm |
| Field moment, | MMSL | = | 1/24 * QMS * s2 | = | 0.83 | kNm |

B. Moment due to additional dead load (MA):

| | | | | | | |
|---|---|---|---|---|---|---|
| Moment of support, | MMAT | = | 5/48 * QMA * s2 | = | 1.12 | kNm |
| Field moment, | MMAL | = | 5/96 * QMA* s2 | = | 0.56 | kNm |

C. Moment due to truck load (TT):

| | | | | | | |
|---|---|---|---|---|---|---|
| Moment of support, | MTTT | = | 5/32 * PTT * s | = | 45.70 | kNm |
| Field moment, | MTTL | = | 9/64 * PTT * s | = | 41.13 | kNm |

D. Moment due to wind load (EW):

| | | | | |
|---|---|---|---|---|
| Service condition load | PEWS | = | 0.741 | Kn |
| Ultimate condition load | PEWU | = | 1,008 | kN |
| Girder spacing | S | = | 2 | m |

| | | | | | | |
|---|---|---|---|---|---|---|
| Service Conditions: | | | | | | |
| Maximum bearing moment, | MEWTS | = | 5/32*PEWS*S | = | 0.22 | kNm |
| Maximum field moment, | MEWLS | = | 9/64*PEWS*S | = | 0.20 | kNm |
| Ultimate condition: | | | | | | |
| Moment of support, | MEWT | = | 5/32 * PEW * s | = | 0.32 | kNm |
| Field moment, | MEWL | = | 9/64 * PEW * s | = | 0.2 | kNm |

E. Moment due to temperature (ET):

| | | | | |
|---|---|---|---|---|
| Elastic modulus, | Ec | = | 25742.96 | MPa |
| Coefficient of expansion, | α | = | 1 x$10^{-5}$ | /ºC |
| Thickness of the floor, | h | = | 200 | mm |
| Temperature difference on the slab | ΔT | = | 12.50 | °C |

Moment of Inertia of concrete floor: I = 1/12 x b x h3
= 1/12 x(500 mm)x(200mm)3 = 33333333.3 mm4

Maximum support moment $M_{ET}$T = 1/4 x ΔT x α x EI/h


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| | | | |
|---|---|---|---|
| | | = | $1/4 \text{ x } 12.50 \text{ x}10^{-5}\text{x}\frac{25742{,}96 \text{ x } 333333333.3}{200}$ |
| | | = | 1.34 kNm |
| Maximum field moment | $M_{ET}$L | = | 7/8 x ΔT x α x EI/h |
| | | = | $7/8 \text{ x } 12.50 \text{ x}10^{-5}\text{x}\frac{25742{,}96 \text{ x } 333333333.3}{200}$ |
| | | = | 4.69kNm |

## 5. Slab Moment Combination

**Table 4. Combination of floor slab moments**

| No | Load Type | Factor burden | Power service | Ultimate state | Field M (kNm) | M. support (kNm) |
|---|---|---|---|---|---|---|
| 1 | Own burden | KMS | 1.00 | 1.30 | 0.83 | 1.67 |
| 2 | Additional dead load | KMA | 1.00 | 2.00 | 0.56 | 1.12 |
| 3 | "T" truck load | Summit | 1.00 | 1.80 | 41.13 | 45.70 |
| 4 | Effect of temperature | KET | 1.00 | 1.20 | 1.34 | 4.69 |
| 5.a | Wind load | KEW | 1.00 | - | 0.20 | 0.32 |
| 5.b | Wind load | KET | - | 1.20 | 0.20 | 0.22 |

Source: Calculation results

### A. Combination – 1 Field Moment

**Table 5. Combinations – 1 Field moments**

| No | Load Type | Load factor | | Field M (kNm) | Service conditions Field MS (kNm) | Ultimate condition Field MU (kNm) |
|---|---|---|---|---|---|---|
| | | Service A | Ultimate B | C | A x C | B x C |
| 1 | Own burden | 1.00 | 1.30 | 0.83 | 0.83 | 1.08 |
| 2 | Additional dead load | 1.00 | 2.00 | 0.56 | 0.56 | 1.12 |
| 3 | "T" truck load | 1.00 | 1.80 | 41.13 | 41.13 | 74.03 |
| 4 | Effect of temperature | 1.00 | 1.20 | 1.34 | 1.34 | 1.61 |
| 5.a | Wind load | 1.00 | - | 0.20 | - | - |
| 5.b | Wind load | - | 1.20 | 0.20 | - | - |
| | Total moment = | | | | 43.86 | 77.84 |

Source: Calculation results

### B. combination – 1 Support Moment

**Table 6. Combination – 1 Support moment**

| No | Load Type | Load factor | | M.support (kNm) | Service conditions MS fulcrum (kNm) | Ultimate condition MU support (kNm) |
|---|---|---|---|---|---|---|
| | | Service A | Ultimate B | C | A x C | B x C |
| 1 | Own burden | 1.00 | 1.30 | 1.67 | 1.67 | 2.17 |
| 2 | Additional dead load | 1.00 | 2.00 | 1.12 | 1.12 | 2.24 |
| 3 | "T" truck load | 1.00 | 1.80 | 45.70 | 45.70 | 82.26 |
| 4 | Effect of temperature | 1.00 | 1.20 | 4.69 | 4.69 | 5.63 |
| 5.a | Wind load | 1.00 | - | 0.32 | - | - |
| 5.b | Wind load | - | 1.20 | 0.22 | - | - |
| | Total moment = | | | | 53.18 | 93.30 |

Source: Calculation results

### C. Combination – 2 Field Moments

**Table 7. Combination – 2 Field moments**

| No | Load Type | Load factor | | M. field (kNm) | Service conditions Field MS (kNm) | Ultimate condition Field MU (kNm) |
|---|---|---|---|---|---|---|
| | | Service A | Ultimate B | C | A x C | B x C |
| 1 | Own burden | 1.00 | 1.30 | 0.83 | 0.83 | 1.08 |
| 2 | Additional dead load | 1.00 | 2.00 | 0.56 | 0.56 | 1.12 |
| 3 | "T" truck load | 1.00 | 1.80 | 41.13 | 41.13 | 74.03 |
| 4 | Effect of temperature | 1.00 | 1.20 | 1.34 | 1.34 | - |
| 5.a | Wind load | 1.00 | - | 0.2 | - | - |
| 5.b | Wind load | - | 1.20 | 0.2 | - | - |
| | Total moment = | | | | 43.86 | 76.23 |

Source: Calculation results

### D. Combination – 2 Support Moments


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**Table 8. Combination – 2 Support Moments**

| No | Load Type | Load factor | | M.support (kNm) C | Service conditions MS fulcrum (kNm) A x C | Ultimate condition MU support (kNm) B x C |
|---|---|---|---|---|---|---|
| | | Service A | Ultimate B | | | |
| 1 | Own burden | 1.00 | 1.30 | 1.67 | 1.67 | 2.17 |
| 2 | Additional dead load | 1.00 | 2.00 | 1.12 | 1.12 | 2.24 |
| 3 | "T" truck load | 1.00 | 1.80 | 45.70 | 45.70 | 82.26 |
| 4 | Effect of temperature | 1.00 | 1.20 | 4.69 | 4.69 | - |
| 5.a | Wind load | 1.00 | - | 0.32 | - | - |
| 5.b | Wind load | - | 1.20 | 0.22 | - | - |
| | Total moment = | | | | 53.18 | 86.67 |

Source: Calculation results

## Slab Reinforcement

### 1. Negative Flexural Reinforcement

Moment of support plan Your = 93.30 kNm
Concrete quality: K – 350 Concrete compressive strength, fc' = 30 MPa
Steel quality: U – 42 Steel yield strength fy = 420 MPa
Thickness of concrete slab, h = 200 mm
The distance between the reinforcement and the outside of the concrete, d' = 30 mm
Modulus of elasticity of steel, Ice = 2 x 105
Concrete stress distribution shape factor, $\beta_1$ = 0.85

$\rho_b$ = $\beta_1$x 0.85 x fc'/ fy x 600 / ( 600 + fy )
= 0.85 x 0.85 x 30 / 420 x 600 / ( 600 + 420 ) = 0.030

$R_{max}$ = 0.75 x$\rho_b$x fy x [ 1 – 1/2 x 0.75 xx fy/ ( 0.85 x fc' )$\rho_b$
= 0.75 x 0.030x 420 x [ 1 – 1/2 x 0.75 x 0.030x 420/(0.85 x 30) = 7,699

Flexural strength reduction factor, ϕ = 0.80
The moment of the ultimate plan, $M_u$ = 93.30 kNm
Effective thickness of concrete slab, d = h – d' = 170 mm
Consider a 1 m wide concrete slab, b = 1000 mm
Nominal moment of plan, $M_n$= $/M_u$φ = 67,354 kNm
Moment resistance factor,
$R_n$ = x 10-6/( bx d2 )$M_n$
= 56.657 x 10-6 / ( 1000 x 1702 ) = 2,331
$R_n$ <$R_{max}$ ( OK )

Required Reinforcement Ratio:
ρ = 0.85 x fc'/ fy x [ 1 -$\sqrt{[1-2\,x\ Rn/(0.85\,x\,fc')}$
= 0.85 x 30 / 420 x [1- $\sqrt{[1-2\,x\ 2{,}331/(0.85\,x\,30)}$ = 0.00583
Minimum reinforcement ratio, $\rho_{min}$ = 25% *(1.4 / fy) = 0.00083

The reinforcement ratio used, P = 0.0058
The required reinforcement area, $A_S$ = ρ x b x d
= 0.0058 x 1000 x 170 = 991.10 mm2
Diameter of reinforcement used D = 16 Mm
The required reinforcement spacing,
s = π/4 x D2 xb /$A_S$
= 3.14/4 x 162 x 1000/991.10 = 202.76 Mm
Used reinforcement, D 16 - 150
$A_S$ == π/4 x D2 xb / s
3.14/4 x 162 x 1000/150 = 1339.73 mm2
Reinforcement for longitudinal shrinkage is taken as 50% x main reinforcement.
$A_S'$ == 50% x$A_s$
50% x 1339.73 = 669.87 mm2
Diameter of reinforcement used, D = 16 Mm
The required reinforcement spacing,
S == π/4 x D2 xb/$A_S$
3.14/4 x 162 x 1000/1339.73 = 150.00 Mm
Used reinforcement, D 16 - 100
$A_S'$ = π/4 x D2 x b/s
= 3.14/4 x 162 x 1000/100 = 2009.6 mm
D 16 – 150 reinforcement is used


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## 2. Positive Flexural Reinforcement

Field planning moment, $M_U$ = 77.84 kNm
Concrete quality: K - 350 Concrete compressive strength, fc' = 30 MPa
Steel quality: U - 42 Steel yield strength fy = 420 MPa
Thickness of concrete slab, h = 200 mm
The distance between the reinforcement and the outside of the concrete, d' = 30 mm
Modulus of elasticity of steel, Ice = 2 x 105
Concrete stress distribution shape factor, $\beta_1$ = 0.85

$\rho_b$ = $\beta_1$x 0.85 x fc'/ fy x 600 / ( 600 + fy ) = 0.030
$R_{max}$ = 0.75 x$\rho_b$x fy x [ 1 – 1/2 x 0.75 xx fy/ ( 0.85 x fc' )$\rho_b$
= 0.75 x 0.030 x 420 x [ 1 – 1/2 x 0.75 x 0.030 x 420 / (0.85 x 30 )
= 7,768

Flexural strength reduction factor, ϕ = 0.80
The moment of the ultimate plan, $M_u$ = 77.84 kNm
Effective thickness of concrete slab, d = h – d' = 170 mm
Consider a 1 m wide concrete slab, b = 1000 mm
Nominal moment of plan, $M_n$= $M_u$/φ = 97.30 kNm
Moment resistance factor,
$R_n$ = $M_n$x 10-6/( b x d2 )
= 97.30 x 10-6 / ( 1000 x 1702 ) = 3.36
$R_n$ < $R_{max}$ ( OK )

Required reinforcement ratio:
ρ = 0.85 x fc'/ fy x [ 1 -$\sqrt{[1-2\,x\ Rn/\,(0.85\,x\,fc')}$
= 0.85 x 30 /420 x [1-$\sqrt{[1-2\,x\,2{,}436/(0.85\,x\,30)}$ = 0.00611
Minimum reinforcement ratio, $\rho_{min}$ = 25% x ( 1.4/fy ) = 0.00083
The reinforcement ratio used, ρ = 0.00611
The area of reinforcement used, $A_S$ = ρ x b x d = 1038.70 mm2
Diameter of reinforcement used, D 16 mm
The required reinforcement spacing, S = π/4 x D2 x b/$A_S$ = 193.47 mm

Used reinforcement, D 16 - 150
$A_S$ = π/4 x D2 x b/s = 1339.73 mm2
Reinforcement for longitudinal shrinkage is taken as 50% of the main reinforcement.
$A_S'$ = 50% x$A_s$
= 50% x 1339.73 = 669.87 mm2
Diameter of reinforcement used, 16 mm
The required reinforcement spacing, S = π/4 x D2 x b/$A_S$ = 150.00 mm2
Used reinforcement, D 16 - 100

$A_S$ = π/4 x D2 x b/s
= 3.14 / 4 x 162 x 1000 / 100 = 2009.6 mm2

**Slab Deflection Control**

Concrete quality: K - 350 Concrete compressive strength, fc' = 30 MPa
Steel quality: U - 42 Steel yield strength fy = 420 MPa
Modulus of elasticity of concrete, Ec = 4700 x$\sqrt{fc'}$ = 25742.96 MPa
Modulus of elasticity of steel, Ice = 2.00 x 105
Slab thickness, h = 200 mm
The distance between the reinforcement and the outside of the concrete, d' = 30 mm
Effective thickness of slab, d = h – d' = 170 mm
Slab reinforcement area, $A_S$ = 1339.73 mm2
Slab span length, Lx = 1.40 m = 1400 mm
Viewed slab width, b = 1.00 m = 1000 mm
Centralized load, P = $T_{TT}$ = 146.25 kN
Evenly distributed load, Q = $P_{MS}$+$P_{MA}$ = 7,690 kN/m
The total deflection that occurs ($\delta_{tot}$) must be < / 240$L_X$ = 5,833 mm
Gross inertia of the plate cross section,
$I_g$ = 1/12 xbx h3
= 1/12 x 1000 x 2003 = 6.67 x 108 mm3
$f_r$ = 0.7 x$\sqrt{fc'}$
= 0.7 x$\sqrt{30}$ = 3,834 MPa
The comparative value of elastic modulus,
n = Es /Ec
= 2x105/23452.45 = 7.77
nx$A_S$ = 7.77 x 1339.73 = 10409.70 mm2
Distance of the neutral line to the outer side of the concrete,
c = nx$A_S$/ b
= 7.77 x 1339.73 / 1000 = 10.41 mm


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The inertia of the cracked cross-section transformed into concrete is calculated as follows.

$I_{cr}$ = 1/3 xbx c3 + nx$A_S$x ( d – c )2
= 1/3 x 1000 x 10.413 + 10409.70 x (170 – 10.41)2 = 2.66 x 108 mm4
$y_t$ = h/ 2
= 200 / 2 = 100 mm
Fracture moment: $M_{cr}$ = $f_r$x / $I_g y_t$
= 3,834 x 6.67 x 108 / 100 = 2.56 x 107 Nmm
Maximum moment due to load (without load factor):
$M_a$ = 1/8 x Q x$L_x{}^2$+ 1/4 x P x $L_x$
= 1/8 x 7,690 x 1.402 + 1/4 x 146.25 x 1.40 = 5.31 x 107 Nmm
Effective inertia for deflection calculations,
$I_e$ = ($M_{cr}$/ )3 x + [ 1 –$M_a I_g$($M_{cr}$/ )3 ] x $M_a I_{cr}$
= ( 2.56 / 5.31 )3 x 6.67 + [ 1 – ( 2.56 / 5.31 )3 ] x 2.66 = 3.11 x 108 mm4
Q = 7690 N/mm
P = 146250 N

Instantaneous elastic deflection due to dead load and live load:
$\delta_e$ = 5/384 x Q x$L_x{}^4$/ ( Ec x ) + 1/48 x P$I_e$x$L_x{}^3$/ ( Ec x )$I_e$
= 5/384 x 7690 x 1,404 / (25742.96 x 3.12 ) + 1/48 x 146250 x 1,403 / (25742.96 x 3.12 )
0.00478921337 + 0.104
= 0.1088 mm
Bridge floor slab reinforcement ratio:
ρ = $A_S$/ ( b x d )
= 1339.73 / ( 1000 x 170 ) = 0.00788
Time dependency factor for dead load (period > 5 years), value:
ζ = 2.00
λ = ζ / ( 1 + 50 x ρ )
= 2.00 / ( 1 + 50 x 0.00788 ) = 1.4347
Long-term deflection due to creep and shrinkage:
$\delta_g$ = λ x 5/384 x Q x$L_x{}^4$/ ( Ec x )$I_e$
= 1.4347x 5/384 x 6590x 1.404 / (25742.96 x 3.12) = 0.0687 mm
Total deflection on the bridge floor: $L_x$/ 240 = 5,833 mm
$\delta_{tot}$ = $\delta_e$+ $\delta_g$
= 0.1088 + 0.0687 = 0.1775 mm
< Lx/240 ( safe ) OK

**Punch Shear Stress Control**

| | | | |
|---|---|---|---|
| Concrete quality: K - 350 | Concrete compressive strength, fc' = | 30.00 | MPa |
| The specified shear strength of the punch, | $f_v$ = 0.3 x$\sqrt{fc'}$ = | 1.64 | MPa |
| Shear strength reduction factor, | ϕ = | 0.60 | |
| Truck wheel load on slab, | $P_{TT}$ = 146.25 kN = | 146250 | N |
| | h = | 1.20 | m |
| | a = | 0.30 | m |
| | $t_a$ = | 0.10 | m |
| | b = | 0.50 | m |

u = a + 2 x$t_a$+ h
= 0.30 + 2 x 0.10 + 0.20 = 0.70 m = 700.00 mm
v = b + 2 x$t_a$+ h
= 0.50 + 2 x 0.1 + 0.20 = 0.90 m = 900.00 mm
Effective thickness of the plate, d = 170.00 mm
Area of shear plane: $A_v$= 2 x ( u + v ) x d = 544000 mm2

Nominal punch shear force, $P_n = A_v$x$f_v$ = 908560 N
Φ x$P_n$ = 545136 N
ultimate load factor, $K_{TT}$ = 2.0
Ultimate load of truck wheels on slab,
$P_u$ = $K_{TT}$x $P_{TT}$
= 2.0 x 146250 = 292500 N
< Φ x$P_n$ SAFE (OK)

## T-Beam Bridge Girders Calculation

## 1. Superstructure Data

| | | | | |
|---|---|---|---|---|
| Bridge span length | | L = | 30.00 | m |
| Road width (traffic lane) | | B1 = | 9.00 | m |
| Sidewalk width | | B2 = | 1.00 | m |
| Total width of the bridge | | B1 + 2 * B2 = | 11.00 | m |
| Distance between Girders | | s = | 1.50 | m |
| Girder Dimensions: | Girder width | b = | 1.00 | m |
| | Girder height | h = | 2.00 | m |
| Diaphragm Dimensions: | Diaphragm width | bd = | 0.30 | m |


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| | | | |
|---|---|---|---|
| Diaphragm height | hd = | 0.70 | m |
| Thickness of bridge floor slab | ts = | 0.20 | m |
| Thickness of asphalt layer + overlay | ta = | 0.10 | m |
| Height of rainwater puddles | th = | 0.05 | m |
| Side plane height | ha = | 2.50 | m |

## 2. Structural Materials Concrete Quality K – 300

Concrete compressive strength, f c' = 0.83 * K / 10 = 24.90 MPa
Elastic modulus Ec = 4700 * Ö fc' = 23452.953Mpa
Poisson's number u = 0.20
Shear modulus G = Ec / [2*(1 + u)] = 9772 MPa
Coefficient of linear expansion for concrete α = 0.00001$^0$C

### Steel Quality

| | | | |
|---|---|---|---|
| For reinforcing steel with Ø > 12 mm: | U - | 39 | |
| Yield stress of steel, | fy = U*10 = | 390 | Mpa |
| For reinforcing steel with Ø ≤ 12 mm: | U - | 24 | |
| Yield stress of steel, | fy = U*10 = | 240 | Mpa |

### Specific Gravity

Weight of reinforced concrete, toilet = 25.00kNm
Weight of unreinforced concrete (rebate concrete) w'c = 24.00 kNm
Solid asphalt weight wa = 22.00kNm
Specific gravity of water www = 9.80 kNm

## 3. Load Analysis

### A. Self Weight (MS)

Ultimate load factor $K_{MS}$= 1.25

Self weight is the weight of the materials and parts of the bridge which are structural elements, plus the non-structural elements that it supports and are permanent. The self weight load of the diaphragm beam on the girder is calculated as follows:

Girder span length, L = 30.00 m
Weight of one diaphragm block Wd = bd* (hd – ts) * s * wc = 5.625 kN
The number of diaphragm beams along the span L, nd = 8 pcs
Diaphragm load on Girder, Qd = nd * Wd / L = 1.5 kN/m

**Table 9. Self-weight load on girder**

| No. | Type | Wide (m) | Thick (m) | Heavy (kN/m3) | Burden (kN/m) |
|---|---|---|---|---|---|
| 1 | Floor slab | 1.50 | 0.20 | 25.00 | 7.50 |
| 2 | Girder | 1.00 | 1.80 | 25.00 | 45.00 |
| 3 | Diaphragm | | | Qd = | 2.40 |
| | | | | **QMS=** | **54.00** |

Source: Data Processing, 2024

Shear force and moment on T-Girder due to self-weight (MS):
VMS = 1/2 * QMS * L = 810,000kN
MMS = 1/8 * QMS * $L^2$ = 6075.00 kN

### B. Additional Dead Load

Ultimate load factor $K_{MA}$= 1.25

Superimposed dead load is the weight of all materials that create a load on the bridge which is a non-structural element, and may change in size during the life of the bridge. The bridge analyzed must be able to carry additional loads such as:

1) Adding an asphalt layer (overlay) at a later date,


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2) Rainwater pooling if the drainage system is not working properly,

Girder span length, L = 30.00 m

**Table 10. Additional dead load on Girder**

| No. | Type | Wide (m) | Thick (m) | Heavy (kN/m3) | Burden (kN/m) |
|---|---|---|---|---|---|
| 1 | Asphalt Lap+overlay | 1.50 | 0.10 | 22.00 | 3.30 |
| 2 | Rainwater | 1.50 | 0.05 | 9.80 | 0.74 |
| Additional dead load: | | | | QMA = | 4.04 |

Source: Data processing, 2024

Shear force and moment on T-Girder due to additional load (MA):
VMA = 1/2 * QMA * L = 60.525 kNm
MMA = 1/8 * QMA * L2 = 453.938 kNm

**C. Traffic Load**
1) Lane Load "D" (TD)
Ultimate load factor $K_{TD}$= 1.8
The vehicle load in the form of a "D" lane load consists of a uniformly distributed load (UDL), UDL and knife edge load (KEL), as shown in Figure 4.8..UDL has an intensity q (kPa) whose value depends on the length of the span L which is loaded with traffic as in Figure 4.9 or is expressed by the following formula:
q = 8.0 kPa for L ≤ 30
q = 8.0 * (0.5 + 15/L) kPa for L > 30

For span length, L = 30.00 m q = 8.00 kPa
KEL has intensity, p = 44.00 kN/m
The dynamic load factor (Dynamic Load Allowance) for KEL is taken as follows:
DLA = 0.40 for L ≤ 50 m
DLA = 0.4 - 0.0025*(L - 50) for 50 < L < 90 m
DLA = 0.30 for L ≥ 90 m

Distance between girders s = 1.50 m
For span length, L = 30.00 m, then DLA = 0.40
Lane load on girder, QTD = q*s = 12.00kN
$P_{TD}$= (1+DLA) * p * q * s = 92.40kN
Shear force and moment on T-Girder due to "D" lane load
VTD = ½ * (QTD * L + PTD) = 226.20 kN
MTD = 1/8 QTD * L2 + ¼ * PTD * L = 2043.00 kN/m

2) Truck Load "T" (TT)
Ultimate load factor: $K_{TT}$= 1.8
The live load on the bridge floor is in the form of a double wheel load by a truck (T load) of which the size is, T = 100 kN
Dynamic load factor for truck loading is taken, DLA = 0.40
"T" truck load PTT = (1 + DLA)*T = 140.00 kN
Girder span length, L = 30.00
Shear force and moment on the T-Gider due to the truck load "T":
VTT = [ 9/8 * L - 1/4 * a + b ] / L * PTT = 175.00kN
MTT = VTT * L/2 - PTT * b = 1925 kNm
The shear force and moment that occur due to traffic loading are taken as having the greatest influence on the T-Girder between the load "D" and the load "T". The maximum shear force due to the load, T VTT = 175.00 kN
Maximum moment due to load, D $M_{TD}$ = 2043.00 kNm


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3) Brake Force (TB)
Ultimate load factor: $K_{TB}$= 1.8
The braking effect of traffic is calculated as a longitudinal force, and is assumed to act at a distance of 1.80 m above the bridge deck. The magnitude of the longitudinal braking force of the bridge depends on the total length of the bridge (Lt) as follows:

| | | |
|---|---|---|
| Brake style, | HTB = 250 | for $Lt \leq 80$ m |
| Brake style, | HTB = 250 + 2.5*(Lt-80) | for $80 < Lt < 180$ m |
| Brake style, | HTB = 500 | for $Lt \geq 180$ m |

Girder span length, L = 30.00 m
Number of girders, ngirder = 5 pcs
Brake style HTB = 250 kN
Distance between girders s = 1.50 m
Brake force for $Lt \leq 80$ m: TTB =HTB / ngirder = 50 kN
The braking force can also be calculated as 5% of the "D" lane load without the dynamic load factor.
Brake style, TTB = 50% of lane "D" load without dynamic load factor

| | | | |
|---|---|---|---|
| QTD | = q * s | = | 12.00kN/m |
| PTD | = p * s | = | 66.00kN |
| TTB | = 0.05 * ( QTD * L + PTD ) | = | 21.30 kN |
| | | < | 50.00 kN |
| TTB | | = | 50.00 kN |

The arm relative to the center of gravity of the block, y = 1.80 + ta + h/2 = 2.90 m
Moment load due to brake force M = TTB * y = 145.00 kNm
Maximum shear force and moment on the beam due to braking force:
VTB = M / L = 4.83 kN
MTB = 1/2 * M = 145.00 kNm

4) Wind Load
Ultimate load factor : KEW = 1.25
The additional horizontal wind force on the surface of the bridge floor due to the wind load blowing the vehicle on the bridge floor is calculated using the formula:

TEW = 0.0012*Cw*(Vw)2 kN/m2 with, Cw = 1.2
Design wind speed, Vw = 35 m/sec
Additional wind load blowing on the side of the vehicle:
TEW = 0.0012*Cw*(Vw)2 = 1,764 kN/m2
The vertical plane that is blown by the wind is the side plane of the vehicle with a height 2.00 m above the bridge floor h = 2.00 m
Distance between vehicle wheels x = 1.75 m
Load due to wind load transfer to the bridge floor,
QEW = 1/2*h / x * TEW = 1.008 kN/m

Girder span length, L = 30.00 m
Shear force and moment on the girder due to wind load (EW):
VEW = 1/2 * QEW * L = 15,120 kN
MEW = 1/8 * QEW * L2 = 113,400 kNm

5) Effect of Temperature
The shear force and moment on the girder due to the influence of temperature are calculated based on the force arising from temperature movement on the support (elastomeric bearing) with a temperature difference of: DT=25 °C
The coefficient of linear expansion for concrete, α = 1.0°E-05°C
Girder span length, L = 30.00 m


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Shear stiffness of elastomeric bearings, k = 15000 kN/m Movement temperature, d = α * DT * L= 0.0060 m
Force due to movement temperature, FET = k * d = 90.00 kN

Girder Height, h = 1.20 m h = 2.00 m
Eccentricity, e = h / 2 = 0.60 e = h/2 = 1.00 m
Moment due to temperature influence, $M = F_{ET}*e$ =90,000 kNm
Shear force and moment on the girder due to the influence of temperature (ET):
$V_{ET}$= M/L = 3,000 kN
$M_{ET}$= M = 90,000 kNm

6) Earthquake Load
The vertical earthquake force on the girder is calculated using a minimum downward vertical acceleration of 0.10 * g (g = gravitational acceleration) or 50% of the equivalent static horizontal earthquake coefficient can be taken.
Horizontal earthquake load coefficient:
Kh = C * S
Kh = Horizontal earthquake load coefficient,
C = Base shear coefficient for the earthquake area, shaking time, and local soil conditions
S = Structural type factor related to the earthquake energy absorption capacity (ductility) of the
The vibration time of the structure is calculated using the formula: T = 2 * p * Ö [ Wt / ( g * KP ) ]

Wt = Total weight consisting of self weight and additional dead load
KP = structural stiffness which is the horizontal force required to cause one unit of deflection.

g = acceleration due to gravity of the earth, g = 9.81 m/sec2
Total weight in the form of own weight and additional dead load:
Wt = QMS + QMA
Self-weight, QMS = 54.90 kNm
Additional dead load, QMA = 4.40 kNm
Span length, L = 30.00 m
Total Weight Wt = (QMS + QMA) * L = 1741.05 kNm
Girder size b = 50 m h = 2.00 m
Moment of inertia of girder cross section, $I = 1/12 * b * h^3$ = 0.666667 $m^4$
Elastic modulus of concrete, Ec = 23453 MPa
Ec = 23452953 kPa
Girder bending stiffness, $Kp = 48 * Ec * I/L^3$ = 27796 kNm
Vibration time, T = 2*p* Ö [ Wt / (g * KP)] = 0.5059 sec

| | | |
|---|---|---|
| The basic soil conditions are moderate. | | |
| Location of the earthquake area | Region = | 3 |
| Base shear coefficient, | C = | 0.18 |
| For bridge structures with reinforced concrete plastic hinge areas, then | | |
| the structure type factor is calculated using the formula, | S = 1.0 * F | |
| with, F = 1.25 - 0.025 * n and F must be taken ≥ 1 | | |
| F = factor of number, | | |
| n = number of plastic hinges that resist deformation of the structure. | | |
| For the value, n = 1 then: | n = | 1 |
| | F = 1.25 - 0.025 * n = | 1,225 |
| Structure type factors, | S = 1.0 * F = | 1,225 |
| Horizontal earthquake load coefficient, | Kh = C*S = | 0.221 |
| Vertical earthquake load coefficient, | Kv = 50% * Kh = | 0.110 |
| The vertical earthquake coefficient is taken, | Kv = | 0.110 |
| Vertical earthquake force, | TEQ = Kv * Wt = | 191,951 |

Vertical earthquake load, $Q_{EQ}$= TEQ / L = 6,398 kN/m


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Shear force and moment on the girder due to vertical earthquake (EQ):

VEQ = QEQ * L = 95.975 kN

MEQ = 1/8 * QEQ * L2 = 719.815 kNm

## 4. Ultimate Load Combination

Table 11. Load Combination Formula

| No. | Load Type | Factor Burden | Comb-1 | Comb-2 | Comb-3 |
|---|---|---|---|---|---|
| 1 | Self weight (MS) | 1.30 | √ | √ | √ |
| 2 | Additional dead load (MA) | 2.00 | √ | √ | √ |
| 3 | "D" lane load (TD) | 2.00 | √ | √ | √ |
| 4 | Brake force (TB) | 2.00 | √ | √ | |
| 5 | Wind load (EW) | 1.20 | √ | | |
| 6 | Effect of Temperature (ET) | 1.20 | | √ | |
| 7 | Earthquake load (EQ) | 1.00 | | | √ |

Source: SNI 1725-2016

**Table 12. Ultimate load combinations**

| THE ULTIMATE MOMENT COMBINATION | | | | Comb-1 | Comb-2 | Comb-3 |
|---|---|---|---|---|---|---|
| No. | Load Type | Factor | M | Your | Your | Your |
| | | Burden | (kNm) | (kNm) | (kNm) | (kNm) |
| 1 | Self weight (MS) | 1.30 | 6075.00 | 7897.50 | 7897.50 | 7897.50 |
| 2 | Additional dead load (MA) | 2.00 | 453.94 | 907.88 | 907.88 | 907.88 |
| 3 | "D" lane load (TD/TT) | 2.00 | 2043.00 | 4086.00 | 4086.00 | 4086.00 |
| 4 | Brake force (TB) | 2.00 | 72.50 | 145.00 | 145.00 | |
| 5 | Wind load (EW) | 1.20 | 113.40 | 136.08 | | |
| 6 | Effect of Temperature (ET) | 1.20 | 90.00 | | 108.00 | |
| 7 | Earthquake load (EQ) | 1.00 | 719.82 | | | 719.82 |
| | | | | 13172.46 | 13144.38 | 13611.19 |

Source: Calculation Results

**Table 13. Ultimate shear force combinations**

| ULTIMATE SHEAR FORCE COMBINATION | | | | Comb-1 | Comb-2 | Comb-3 |
|---|---|---|---|---|---|---|
| No. | Load Type | Factor | V | Vu | Vu | Vu |
| | | Burden | (kN) | (kN) | (kN) | (kN) |
| 1 | Self weight (MS) | 1.30 | 810.00 | 1053.00 | 1053.00 | 1053.00 |
| 2 | Additional dead load (MA) | 2.00 | 60.53 | 121.05 | 121.05 | 121.05 |
| 3 | "D" lane load (TD/TT) | 2.00 | 175.00 | 350.00 | 350.00 | 350.00 |
| 4 | Brake force (TB) | 2.00 | 4.83 | 9.67 | 9.67 | |
| 5 | Wind load (EW) | 1.20 | 15.12 | 18.14 | | |
| 6 | Effect of Temperature (ET) | 1.20 | 3.00 | | 3.60 | |
| 7 | Earthquake load (EQ) | 1.00 | 95.98 | | | 95.98 |
| | | | | 1551.86 | 1537.32 | 1620.03 |

Source: Calculation results

Ultimate moment of girder plan Mu =13611.19 kN/m

Ultimate shear force of girder design Vu =1620.03 kN

**Girder Reinforcement**

**1. Flexible Reinforcement**

Ultimate design moment of girder, Your = 13611.19 kNm

Concrete quality: K – 300 fc' = 24.9

Reinforcing steel quality U – 39 fy = 390 MPa

Thickness of concrete slab, ts = 200 mm

Girder body width, b = 1000 mm

Girder height, h = 2000 mm

The width of the T-Girder wings is taken as the smallest value. L/4 = 3750 mm

s = 1500 mm

12 * ts = 2400 mm

Take the effective width of the T-Girder wing, beff = 200 mm beff = 1500 mm

Distance between the center of the reinforcement and the outside of the concrete, d' = 150 mm d' = 150 mm

Elastic modulus of steel, Es = 2.00E+05 MPa Ice = 2.0E+05 Mpa

Concrete stress distribution shape factor, β1 = 0.85


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rb = b1* 0.85 * fc'/ fy * 600/(600+fy) = 0.027957
Rmax = 0.75*rb*fy*[1-1/2*0.75*rb*fy/(0.85*fc')] = 6.597664
Flexural strength reduction factor, f = 0.80
Effective height of T-Girder, d = h - d' = 1850 mm
Nominal moment of plan, Mn = Mu/f = 17192.47 kNm
Moment resistance factor, Rn = Mn * 106 / (beff * d2) = 3.34
**Rn < Rmax OK**

Required reinforcement ratio: r =
0.85 * fc' / fy * [ 1 - Ö (1 – 2 * Rn / ( 0.85 * fc' ))] = 0.009294
Minimum reinforcement ratio, rmin = 1.4 / fy = 0.00359
The required reinforcement area, As = r * beff * d = 25789.64 mm2
The diameter of the reinforcement used is D 32 mm
As1 = p/4 * D2 = 804.25 mm2
The amount of reinforcement required, n = As / As1 = 32.44
Used reinforcement, **33 D 32**
As = As1 * n = 26540.17 mm2
Thickness of concrete cover, td = 30 mm
Diameter of stirrup used, etc = 13 mm
Number of reinforcements per row, nt = 6
Clear distance between reinforcement, mm
X = ( b - nt * D - 2 * td - 2 * ds) / (nt - 1) = 144.4 mm
**> 35 mm OK**
To ensure that the girder is ductile, the compression reinforcement is taken as 30% of the reinforcement. pull, so: As' = 30% * As = 7962.052 mm2
Used reinforcement, 6 D 32
As = As1*n = 47772.31 **OK**

## 2. Ultimate Moment Capacity Control

| | | | |
|---|---|---|---|
| Thickness of concrete slab, | ts = | 200 | mm |
| Effective wingspan, | beff = | 2000 | mm |
| Girder body width, | b = | 500 | mm |
| Girder Height, | h = | 1200 | mm |
| The distance between the center of the reinforcement and the outside of the concrete, | d' = | 150 | mm |
| Effective height of T-Girder, | d = h - d' = | 1050 | mm |
| Reinforcement area, | As = | 38603.89 | mm2 |
| Concrete compressive strength, | fc' = | 24.9 | Mpa |
| Steel yield strength, | fy = | 390 | MPa |
| For the neutral line to be inside the T-Girder wing, then: | | Cc > Ts | |
| Internal compressive force of concrete on the flange, | Cc = 0.85 * fc' * beff * ts = | 8466000 | N |
| Internal tensile force of reinforcing steel, | Ts = As * fy = | 15055517.31 | N |

Thickness of concrete slab, ts = 200 mm
Effective wingspan, beff = 1500 mm
Girder body width, b = 1000 mm
Girder Height, h = 2000 mm
The distance between the center of the reinforcement and the outside of the concrete, d' = 150 mm
Effective height of T-Girder, d = h - d' = 1850 mm
Reinforcement area, As = 26540.17 mm2
Concrete compressive strength, fc' = 24.9 Mpa
Steel yield strength, fy = 390 MPa
For the neutral line to be inside the T-Girder wing, then: Cc > Ts


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Internal compressive force of concrete on the flange, Cc = 0.85 * fc' * beff * ts = 6349500 N
Internal tensile force of reinforcing steel, Ts = As * fy = 10350668.15 N
For the neutral line to be inside the T-Girder wing, then: Cc > Ts

Internal compressive force of concrete on the flange,

Cc = 0.85 * fc' * beff * ts = 6349500 N

Ts = As * fy = 10350668 N

Cc > Ts Neutral line inside the wing

a = As * fy / (0.85 * fc' * beff) = 326.03 mm

Neutral line distance, c = a B/1 = 383.57mm

Strain in Tensile reinforcing steel, es = 0.003 * (d - c) / c = 0.0115
< 0.03 Ok

Nominal moment, Mn = As * fy * (da/2) * 10-6 = 17461.417 kNm

Ultimate moment capacity, I * Mn = 3506.390 kNm, Ø*Mn = 13969.13 kNm
Mn > Mu 13753.98 kNm Ok….

**3. Shear Reinforcement**

Design ultimate shear force, Vu = 1620.03 kN
Concrete quality: K – 300 Concrete compressive strength, fc' = 24.9MPa
Reinforcing steel quality: U – 39 Yield strength of steel, fy = 390 MPa
Shear strength reduction factor, f = 0.75
Girder body width, b = 1000 mm
Effective height of girder, d = 1850 mm
Nominal shear strength of concrete, Vc = (√ fc') / 6 * b * d * 10-3 = 1538.580 kN
ϕ * Vc = 1153.935 kN
Shear reinforcement is required

ϕ * Vs = Vu - ϕ * Vc = 466,090 kNm

Shear force carried by shear reinforcement, Vs = 621.454 kN
Girder dimension control for maximum shear strength:

Vsmax = 2 / 3 * Ö fc' * [ b * d ] * 10-3 = 6154.321 kN
Vs < Vsmax

The beam dimensions meet the shear strength requirements, OK…
Used stirrups with cross section: 2 D13
The area of the stirrup shear reinforcement, Av = π/4 * D2 * n =265,465 mm2
Required shear reinforcement (stirrup) spacing:

S = Av * fy * d / Vs = 308.201 mm

Used stirrups, 2 D13-200
On the girder body, minimal shrinkage reinforcement is installed with a reinforcement ratio of,
ρsh = 0.001
Area of shrinkage reinforcement, Ash = ρh * b * d = 1850 mm2
The diameter of the reinforcement used is D 13 mm
The amount of shrinkage reinforcement required, n = Ash / (π /4 * D2 ) = 13.94 Reinforcement used, 15 D13

**4. Beam Deflection**

| | | | | | | |
|---|---|---|---|---|---|---|
| Concrete quality: | K - | 300 | Concrete compressive strength, | fc' = | 24.9 | MPa |
| Reinforcing steel quality: | U - | 39 | Steel yield strength, | fy = | 390 | MPa |
| Elastic modulus of concrete, | | | | Ec = 4700 *√fc' = | 23453 | MPa |
| Elastic modulus of steel, | | | | Ice = | 2.0,E+05 | MPa |
| Height of the beam, | | | | h = | 1.20 | m |
| Beam width, | | | | b = | 0.50 | m |
| The distance between the reinforcement and the outside of the concrete, | | | | d' = | 0.15 | m |
| Effective height of the beam, | | | | d = h - d' = | 1.05 | m |
| Beam reinforcement area, | | | | As = | 0.038604 | m2 |
| Gross inertia of the beam cross-section, | | | | Ig = 1/12 * b * h3 = | 0.072 | m4 |


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| | | | |
|---|---|---|---|
| Flexural modulus of rupture of concrete, | fr = 0.7 *√fc' * 103 = | 3492.992986 | kPa |
| The comparative value of elastic modulus, | n = Es / Ec = | 8.5 | |
| | n * As = | 0.329 | m2 |
| Distance of the neutral line to the top of the concrete, | c = n * As / b = | 0.658 | m |

The inertia of the cracked cross-section transformed into concrete is calculated as follows:

| | | | |
|---|---|---|---|
| | Icr = 1/3 * b * c3 + n * As * ( d - c )2 = | 0.09805 | m4 |
| | yt = h/2 = | 0.60 | m |
| Fracture moment: | Mcr = fr * Ig / yt = | 419,159 | Nmm |

Concrete quality: K -300 Concrete compressive strength, fc' = 24.9 MPa
Reinforcing steel quality: U - 39 Steel yield strength, fy = 390 MPa
Elastic modulus of concrete, Ec = 4700 * Ö fc' = 23453 MPa
Elastic modulus of steel, Ice = 2.0,E+05MPa
Height of the beam, h = 2.00 m
Beam width, b = 1.00 m
Distance of reinforcement from the outside of the concrete, d' = 0.15 m
Effective height of the beam, d = h - d' = 1.85 m
Beam reinforcement area, As = 0.026540 m2
Gross inertia of the beam cross-section, Ig = 1/12 * b * h3 = 0.666666667 m4
Flexural modulus of rupture of concrete, fr = 0.7 * Ö fc' * 103 = 3492.992986 kPa
The comparative value of the elastic modulus, n = Es / Ec = 8.5
n * As = 0.226 m2
Distance of neutral line to the top of concrete, c = n * As / b = 0.226 m
The inertia of the cracked cross-section transformed into concrete is calculated as follows:

Icr = 1/3 * b * c3 + n * As * ( d - c )2 = 0.60053 m4
yt = h/2 = 1.00 m
Cracking moment : Mcr = fr * Ig / yt = 2328,662 Nmm

Moment due to dead load and live load (MD+L)

**Table 14. MDL Moment**

| No. | Load Type | | Moment (kNm) |
|---|---|---|---|
| 1 | Self weight (MS) | | 6075.00 |
| 2 | Additional dead load (MA) | | 453.94 |
| 3 | Traffic load (TD/TT) | | 2043.00 |
| 4 | Brake force (TB) | | 72.50 |
| | | MD+L = | 8745.69 |

Source: Calculation Results

Effective inertia for deflection calculations

Ie =( Mcr / MD+L )3 * Ig + [ 1 - ( Mcr / MD+L )3 ] * Icr = 0.6018 $m^4$

Length of beam span, L = 30 m

a) Deflection due to self-weight (MS)

The burden due to one's own weight, QMS = 54.00 kN/m

Deflection due to self-weight (MS):

dMS = 5/384*QMS*L4 / ( Ec*Ie) = 0.04035 m

b) Deflection due to additional dead load (MA)

The burden due to one's own weight, QMA = 4.04 kN

Deflection due to self-weight (MS)

dMA = 5/384*QMA*L4 / ( Ec*Ie) = 0.00302 m

c) Deflection due to load on lane "D" (TD)

Load "D" concentrated load, $P_{TD}$ = 92.40 kN


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Even load $Q_{TD}$ = 12.00 kN/m
Deflection due to lane “D” (TD)
dTD = 1/48*PTD*L3 / (Ec*Ie) + 5/384*QTD*L4 / ( Ec*Ie) = 0.01265 m
d) Deflection due to braking force (TB)
Moment due to braking force, $M_{TB}$ = 72.50 kN
Deflection due to braking force (TB)
δTB = 0.0642 * MTB * L2 / ( Ec*Ie) = 0.00030
e) Deflection due to wind load (EW)
Load due to wind load transfer on vehicles, $Q_{EW}$ = 1.008kN/m
Deflection due to wind load (EW)
δEW = 5/384*QEW*L4 / ( Ec*Ie) = 0.0008 m
f) Deflection due to temperature (ET)
Moment due to movement temperature, MET = 90.00 kN/m
Deflection due to wind load (EW)
δEW = 0.0642*MET*L2/ ( Ec*Ie) = 0.00037 m
g) Deflection due to temperature influence (EQ)
Vertical earthquake moment $Q_{EQ}$ = 6.398 kN/m
Deflection due to wind load (EQ)
δEQ = 5/384*QEQ*L4 / ( Ec*Ie) = 0.0048 m
Ultimate load combination: δEQ L/240 = 0.125

**Table 15. Load Combinations**

| No. | Load Type | Comb-1 (kNm) | Comb-2 (kNm) | Comb-3 (kNm) |
|---|---|---|---|---|
| 1 | Self weight (MS) | 0.0404 | 0.0404 | 0.0404 |
| 2 | Additional dead load (MA) | 0.0030 | 0.0030 | 0.0030 |
| 3 | "D" lane load (TD/TT) | 0.0126 | 0.0126 | 0.0126 |
| 4 | Brake force (TB) | 0.0003 | 0.0003 | |
| 5 | Wind load (EW) | 0.0008 | | |
| 6 | Effect of Temperature (ET) | | 0.0004 | |
| 7 | Earthquake load (EQ) | | | 0.0048 |
| | | 0.0571 | 0.0567 | 0.0608 |
| | | < L/240 OK | < L/240 OK | < L/240 OK |

Source: Calculation Results

**5. Reinforcement of Diaphragm Beam**
**Load on Diaphragm Beam**

The distribution of floor loads on the diaphragm beam is as follows:
Diaphragm beam size, Width bd = 0.30 m
Height hd = 0.70 m
Floor thickness ts = 0.20 m

**Table 16. Self weight (MS)**

| No. | Type | Wide | Thick | Heavy (kN/m3) | Burden (kN/m) |
|---|---|---|---|---|---|
| 1 | Floor slab | 1.50 | 0.20 | 25.00 | 7.50 |
| 2 | Diaphragm beam | 0.30 | 0.50 | 25.00 | 3.75 |
| | | | | QMS = | 11.25 |

Source: Calculation Results

Shear force and moment due to self-weight:
VMS = 1/2 * QMS * s = 8.438 kN
MMS = 1/12 * QMS * s2 = 2.109 kN/m


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**Table 17. Additional dead load (MA):**

| No. | Type | Wide | Thick | Heavy (kN/m3) | Burden (kN/m) |
|---|---|---|---|---|---|
| 1 | Asphalt Lap+overlay | 1.50 | 0.10 | 22.00 | 3.30 |
| 2 | Rainwater | 1.50 | 0.05 | 9.80 | 0.74 |
| | | | | QMS = | 4.04 |

Source: Calculation Results

Shear force and moment due to additional dead load:
VMA = 1/2 * QMA * s = 3.026 kN
MMA = 1/12 * QMA * s2 = 0.757 kN/m
Truck load "T" (TT):
The live load on the bridge floor is in the form of a double wheel load by a truck (T load) of which the size is, T = 100 kN
The dynamic load factor for truck loading is taken, $D_{LA}$ = 0.40
Truck load "T" : PTT = (1 + DLA) * T = 140.00 kN
Shear force and moment due to load "T",
VTT = 1/2 * PTT = 70.00 kN
MTT = 1/8 * PTT * s = 26.25 kN/m

**Table 18. Ultimate load combinations**

| No. | Type of load | Factor Burden | V (kN) | M (kNm) | Vu (kN) | Your (kNm) |
|---|---|---|---|---|---|---|
| 1 | Self weight (MS) | 1.30 | 8.44 | 2.11 | 10,969 | 2,742 |
| 2 | Beb.mati tamb (MA) | 2.00 | 3.03 | 0.76 | 6,053 | 1,513 |
| 3 | Truck load "T" (TT) | 2.00 | 70.00 | 26.25 | 140,000 | 52,500 |
| | | | | | 157,021 | 56,755 |

Source: Calculation Results

Moment and Shear Force of Diaphragm Beam Design
Ultimate moment of diaphragm beam design, Your = 56.755 kN/m
Ultimate shear force of diaphragm beam design, Vu = 157.021 kN

**1. Flexible Reinforcement**

Ultimate design moment of diaphragm beam, Mu = 78,895 kNm Mu = 56.755 kNm
Concrete quality: K-300 Concrete compressive strength, fc' = 24.9 MPa
Reinforcing steel grade: U - 39 Steel yield strength, fy = 390 MPa
Elastic modulus of concrete, Ec = 4700 * Ö fc' = 23453MPa
Elastic modulus of steel, Es = 2.0,E+05 MPa
Beam width, b = bd = 300 mm
Height of the beam, h = hd = 1000 mm
The distance between the center of the reinforcement and the outside of the concrete, d' = 50 mm
Concrete stress distribution shape factor, b1 = 0.85
rb = b1* 0.85 * fc'/ fy * 600/(600+fy) = 0.027957
Rmax = 0.75*rb*fy*[1-1/2*0.75*rb*fy/(0.85*fc')] = 6.597664
Flexural strength reduction factor, f = 0.80
Effective height of the beam, d = h - d' = 650 mm
Nominal moment of plan, Mn = Mu/f =70.94414063 kNm
Moment resistance factor, Rn = Mn * 106 / (beff * d2) = 0.559717086
**Rn < Rmax OK**
Required reinforcement ratio:
r = 0.85 * fc' / fy * [ 1 - Ö (1 – 2 * Rn / ( 0.85 * fc' ))] = 0.001455
Minimum reinforcement ratio, rmin = 1.4 / fy = 0.00359
The required reinforcement area, As = r * b * d = 283.66 mm2
Diameter of reinforcement used, D 15 mm
As1 = p/4 * D2 = 113.10 mm2
The amount of reinforcement required, n = As / As1 = 1.72


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Used reinforcement, 2 D 12
As = As1 * n = 226.195 mm2

**2. Shear Reinforcement**

Design ultimate shear force, Vu = 157.02 kN
Concrete quality: K-300 Concrete compressive strength, fc' =24.9 MPa
Reinforcing steel grade: U-39 Yield strength of steel, fy = 390 MPa
Shear strength reduction factor, f = 0.75
Girder body width, b = 300 mm
Effective height of girder, d = 650 mm
Nominal shear strength of concrete, Vc = (√ fc') / 6 * b * d * 10-3 = 162.175 kN
f * Vc = 121.631 kN
Shear reinforcement is required f * Vs = Vu - f * Vc = 35.390 kN
Shear force carried by shear reinforcement, Vs = 47.187 kN
Girder dimension control for maximum shear strength:
Vsmax = 2 / 3 * Ö fc' * [ b * d ] * 10-3 = 648,699 kN
Vs < Vsmax
The beam dimensions meet the shear strength requirements, OK….
Used stirrups with cross section: 2 D12
Area of shear reinforcement of stirrups, Av = p/4 * D2 * n = 226.195mm2
Distance of shear reinforcement (stirrups) required: S = Av * fy * d / Vs = 1215.173 mm
Stirrups are used, 2 D 12 - 200

## Conclusion

This study found that the calculation of the upper structure of the Wai Poka Bridge which was replaced from a steel truss to a conventional concrete girder with a span of 30 meters meets the SNI 1725-2016 standard. The maximum ultimate moment reaches 13,611 kNm on the girder and a shear force of 1,620 kN, with reinforcement such as 33D32 for bending and 2D13-200 stirrups ensuring safe capacity against dead, live, wind, temperature, and earthquake loads. All deflection, shear, and ductility controls are met, making the existing design economical and reliable for national traffic in Maluku.

However, the study's limitations lie in its quantitative descriptive approach based on existing data without laboratory testing or advanced dynamic simulations, and its assumption of moderate soil conditions without specific geotechnical data. Suggestions for future research include 3D finite element analysis and post-construction monitoring. Practical implications include design guidelines for similar bridge rehabilitation in earthquake-prone areas, supporting PUPR budget efficiency and transportation safety in Eastern Indonesia.

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