1.0 Standards and References

The following main standards shall be standard to be considered for design.

- **General Guidelines /norms**

◊ Basis of structural design;	Eurocodes
◊ Actions on structures	Eurocodes
◊ Concrete structures design specifications,	Eurocodes
◊ Geotechnical design specifications;	Eurocodes
◊ Earthquake resistant structures requirements;	Eurocodes
◊ Road standards	Nepal Road Standard 2070

- **Guidelines/norms for tunnel design**

◊ Electrical standard;	Eurocodes /Swiss Guidelines
◊ Road signalling standards;	Traffic Safety Manuals of DoR
◊ Tunnel specifications;	Eurocodes /Swiss Guidelines
◊ Road tunnel safety systems specification.	Eurocodes /Swiss Guidelines

- **Minimum Safety Requirements for Road Tunnels as per Explained un Employer's Requirements**

The Project standards adopted for design are detailed in the following paragraphs.

- Nepal Road Standard – 2070, 2013
- Nepal Standard Specifications for Road and Bridge works – 2073, 2013
- IRC:37-2018 :Guidelines For The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS
- Nepal Safety Barrier, 1997
- Nepal Delineation Measures, 1996
- Nepal Design Safe Side Drains, 1996
- Nepal Traffic Sign Manual, 1997
- EN 1990:2002, Eurocode 0: Basis of structural design.
- EN 1991 - all parts, Eurocode 1: Actions on structures.
- EN 1992-1-1:2004, Eurocode 2: Design of concrete structures. General rules and rules for buildings.
- EN 1992-2:2005, Eurocode 2: Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules
- EN 1997-1:2004, Eurocode 7: Geotechnical design. General rules.
- EN 1998-1:2004, Eurocode 8: Design of structures for earthquake resistance. General rules, seismic actions and rules for buildings.
- EN 1998-2:2005, Eurocode 8: Design of structures for earthquake resistance – Part 2: Bridges

- ◊ EN 1998-5:2004, Eurocode 8: Design of structures for earthquake resistance. Foundations, retaining structure and geotechnical aspects.
- ◊ ASTRA 12006 (2008), effects of stone chips on protective galleries.
- ◊ ASTRA 11001 (2017), guideline: normal profiles. Federal Roads Office.
- ◊ ASTRA 13001 (2008), Guidelines: Ventilation of the road tunnels.
- ◊ ASTRA technical manual for operational and safety equipment 23 001.
- ◊ ASTRA technical manual tunnel / geotechnical 24 001.◊
- ◊ SIA 197 (2004), project planning tunnel, basics.
- ◊ SIA 197/2 (2004), project planning tunnel, road tunnel.
- ◊ ASTRA 12006 (2008), effects of rockfalls on protective galleries (rockfall load) [as applicable]
- ◊ VSS 640 202 (2017) – Road geometrical normal profile.
- ◊ CEB N°187 - Comité-Euro-International du Béton (1988). Concrete Structures under Impact and Impulsive Load, Bulletin d'information N° 187, Draft TG V/14, Lausanne.
- ◊ EOTA (2012). ETAG 027 – Falling rock protection kits. Guideline for european technical approval.
- ◊ EN 206:2016, Concrete – Specification, performance, production and conformity
- ◊ EN 197-1:2011, Cement – Part 1: Composition, specifications and conformity criteria for common cement
- ◊ EN 12620:2008, Aggregates for concrete
- ◊ EN 1537– Execution of special geotechnical works - Ground anchors
- ◊ EN ISO 22477-5:2018 - Geotechnical investigation and testing - Testing of geotechnical structures - Part 5: Testing of grouted anchors

Swiss Federal road administration (FEDRO) directives for design of tunnel operating and safety systems (***supplement to Minimum Safety Requirements for Road Tunnels as per Explained un Employer's Requirements***):

- ◊ FEDRO 13001 ventilation of the road tunnels (2008 V2.03)
- ◊ FEDRO 13002 ventilation of the safety tunnels of road tunnels (2008 V1.06)
- ◊ FEDRO 13004 fire detection in road tunnels (2007 V2.10)
- ◊ FEDRO 13005 video systems (2012 V1.01)
- ◊ FEDRO 13006 radio systems in road tunnels (2018 V5.00)
- ◊ FEDRO 13010 signaling of safety devices in road tunnels (2011 V2.06)
- ◊ FEDRO 13011 doors and gates in road tunnels (2009 V1.05)
- ◊ FEDRO 13015 lighting systems (2017 V1.11)

Standard for tunnel ventilation system

- ◊ Road Tunnels: Vehicle Emissions and Air Demand for Ventilation, Technical Committee D.5 Road Tunnels, PIARC, World Road Association, 2018R038EN
- ◊ Swiss Tunnel Ventilation Design Code (Federal Roads Office "FEDRO"; Richtlinie Lüftung der Strassentunnel, Systemwahl, Dimensionierung und Ausstattung), V2.03, 2008
- ◊ Jet fans in portals and in niches, arrangement and efficiency, CFD, Franz Zumsteg, 2007

2.0 Recommended Software

- ◊ MIDAS/Civil 2019, Ver. 1.1. Developed by MIDAS Information Technology Co., Ltd. (Seongnam, Gyeonggi-do, 463-824, Korea).
- ◊ Plaxis 2D 2019, Plaxis bv, The Netherlands
- ◊ Unwedge, Rockscience, Toronto
- ◊ Larix 7, Cubus AG, Zürich
- ◊ Statik 7, Cubus AG, Zürich
- ◊ Fagus 7, Cubus AG, Zürich
- ◊ WXDEF, for section checks
- ◊ ecorisQ Rockyfor3D
- ◊ 2D RocFall
- ◊ Elastic Multi – Layer software KENPAVE (University of Kentucky)

3.0 Input Data Provided by the Client

- ➢ Road Width
- ➢ Traffic Data
- ➢ Precipitation Data
- ➢ Temperature Data
- ➢ Wind Data

4.0 Geotechnical Characterization and Rockfall Hazard

4.1 Geotechnical and geomechanical characterization of the rock mass

Tunnel alignment : The tunnel shall be divided into homogeneous sections ("TO"). Each section shall be characterized by specific rock mass parameters (such as RMR,Q-Value, RQD, GSI, Jv, Jc, ecc.), as well as geotechnical parameters according to the chosen failure criterion, joint parameters and permeability values.

Note : While preparing the section, Geotechnical Design Memorandum (GDM) based Geotechnical Data Report (GDR) shall be used.

Road alignment : The road shall be divided into homogeneous sections ("TO"). Each section shall be characterized by specific rock mass parameters (such as RMR,Q-Value, RQD, GSI, Jv, Jc, ecc.), as well as geotechnical parameters according to the chosen failure criterion, joint parameters and permeability values.

4.2 Tunnel hazard scenarios

The prognosis hazard scenarios shall be assessed. At least the following hazards shall be evaluated :

- ◊ Rock falling/wedges
- ◊ Shear zone and fault gouge
- ◊ Detachment of slabs
- ◊ Tunnel face instability
- ◊ Loosening
- ◊ Plastic deformation
- ◊ Rockburst
- ◊ Water inflow
- ◊ Gas fields

4.3 Rockfall hazard evaluation (if applicable)

4.3.1 Evaluation of rockfall hazard probability

3D analyses and simulations to detect the trajectories for the rockfall process shall be performed, in order to evaluate all areas subjected to rockfall risk, based on a detailed topographic 3D model of the whole mountain side (from the top to the river) where the existing road is located.

4.3.2 Design rock block dimension

Characteristic and, hence, design rock block dimension must be evaluated, based on field data (geological mapping and recording, analysis of orthophotos,…) and statistic analysis of recorded block sizes along the existing road.

4.3.3 Design rockfall energy

Based on 2D/3D rockfall simulations in different characteristic sections along the road alignment and on the assumed design boulder volume, the design impact energy for rock protections must be estimated along the road alignment. For the rockfall simulation, proper surface parameters must be assumed based on proper site surveys.

4.4 Hydrogeology

Hydrogeology of the project area must be investigated, considering that:

- o The morphology presents multiple streams that run along the slope, whose water flow rate varies greatly depending on the season.
- o During periods of heavy and persistent rainfall, the presence of a ground water circulation at the interface bedrock - soil cover can be expected.
- o Suspended aquifers and water pockets in the rock mass are possible, locally and/or seasonally.
- o The conditions of the joints, in particular the opening and the interlocking degree of the rock mass, are the determining factors for water circulation in the bedrock.

5.1 Materials

The main properties of the materials foreseen within the project are detailed in the following paragraphs for the different specific civil works and then summarized in table within paragraph 5.1.7.

5.1.1 Tunnel

5.1.1.1 Concrete

Temporary Shotcrete (Tunnel) for rock excavation support

- Reference Code: UNI EN 1992 1-1, EN 206:2016
- Concrete Class: C25/30
- fck = 25 MPa
- Cement Type: III-IV (EN 197-1)
- Exposure Class: XC2
- Slump Class: S4
- Maximum Aggregate: 8mm (EN 12620)
- Steel fibres (where foreseen):
  - Diameter/Length: 0.55 mm/33 mm
  - L/D aspect ratio: 60
  - Tensile strength: > 1100 MPa
  - Elongation at break: < 4%
  - Dosage: 50 kg/m3

Permanent shotcrete for Portal Excavation Support

- Reference Code: UNI EN 1992 1-1, EN 206:2016
- Concrete Class: C30/37
- Cubic compressive strength Rck ≥ 37 MPa
- Cement Type: III-IV (UNI EN 197-1)
- Exposure Class: XC4 and XD1
- Alkali Aggregate Reaction (AAR) resistant
- Slump Class: S4
- Maximum Aggregate: 16mm (EN 12620)

Concrete for inner lining

- Concrete class, C30/37
- Exposure classes XC4, XD1 and XF1
- Maximum aggregate diameter, 32 mm
- Cubic compressive strength Rck ≥ 37 MPa
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

Lean Concrete

- Concrete class, C12/15
- Cubic compressive strength Rck ≥ 15 MPa
- Min. cement content: 150kg/m^3

Cement-Based Grout For Rock bolts/Anchors Injection

- Type: High strength, non-shrink cementitious construction grout with aggregate size suitable for the effective pouring thickness
- Plasticizer: 1% - 2% on weight
- Strength: 5MPa after 48h, 25MPa after 28d

5.1.1.2 Steel

Steel for reinforcement

a. STEEL RIBBED BARS:

- Bar diameter, D = according to calculations (between 14 mm and 22 mm)
- Steel grade B500 B
- Yield Strength, fyk ≥ 500 MPa

b. WIRE MESH

- Bar diameter, D = 5mm
- Steel grade min. B500 A
- Yield Strength, fyk ≥ 500 MPa

Structural Steel

a. STEEL RIBS: for excavation support

- Profile: HEB 180
- Steel grade S355
- Yield Strength, fyk ≥ 355 MPa

b. LATTICE GIRDERS for tunnel excavation support

- Profile: according to tunnel support classes
- Type: JB-3-Gurt or similar
- Steel grade S235; S500
- Yield Strength, fyk ≥ 235MPa; 500 MPa

Rock bolts

a. ROCK BOLTS, SWELLEX PM 16 for tunnel rock excavation support

- Tensile Strength Ftk ≥ 160 kN
- Yield Strength, Fyk ≥ 130 kN
- Drilling diameter ≥ 50 mm

b. ROCK BOLTS, SWELLEX PM 24 for tunnel rock excavation support

- Tensile Strength Ftk ≥ 240 kN
- Yield Strength, Fyk ≥ 200 kN
- Drilling diameter ≥ 50 mm

c. CEMENTED ROCK BOLTS, M33 for tunnel rock excavation support

- Tensile Strength Ftk ≥ 460 kN

- Yield Strength, Fyk ≥ 364 kN
- Type: Belloli Belcem M33 or similar
- Drilling diameter ≥ 60 mm

d. CEMENTED ROCK BOLTS, M27 for tunnel rock excavation support

- Ftk ≥ 350 kN
- Yield Strength, Fyk ≥ 280 kN
- Type: Belloli Belcem M27 or similar
- Drilling diameter ≥ 60 mm

e. CEMENTED ROCK BOLTS, M24 for tunnel rock excavation support

- Tensile Strength Ftk ≥ 250 kN
- Yield Strength, Fyk ≥ 200 kN
- Type: Belloli Belcem M24 or similar
- Drilling diameter ≥ 60 mm

f. PERMANENT ROCK BOLTS, GEWI BAR 32 for portal excavation support

1. Nominal Diameter : 32mm
2. Cross Sectional Area: 804mm2
3. Steel grade: S500/550
4. Yield Strength, Fyk≥500 MPa
5. Tensile Strength, Ftk ≥550 MPa
6. Load at yield Py: 402 kN
7. Ultimate load Pt: 442 kN
8. Weight: 6.31 kg/m
9. Other requirements: DCP
10. Weight DCP: 9.50 kg/m
11. Drilling Diameter: 120mm

g. PERMANENT ROCK BOLTS, GEWI PLUS 43, for portal excavation support

1. Nominal Diameter : 43mm
2. Cross Sectional Area: 1452mm2
3. Steel grade: S670/800
4. Yield Strength, Fyk≥ 670 MPa
5. Tensile Strength, Ftk≥ 800 MPa
6. Load at yield: 973 kN
7. Ultimate load: 1162 kN
8. Weight: 11.40 kg/m
9. Other requirements: DCP
    - Weight DCP: 15.80 kg/m
    - Drilling Diameter: 120mm

Steel for Pre-tensioned Anchors

TEMPORARY PRE TENSIONED GROUTED ANCHOR for portal excavation support

- Harmonic Steel: 0.6''
- Steel grade: S1670/1860
- Yield Strength, Fyk≥ 1670 MPa
- Tensile Strength, Ftk≥ 1860 MPa
- Number of strands: 5
- Nominal Strand area, Ai: 140 mm2
- Load at yield, Py: 234 kN/strand
- Ultimate Load, Pt: 260 kN/strand
- Prestress Load: 70 kN/strand
- Drilling Diameter: 200mm

Steel for Forepoling

a. FOREPOLING BARS for rock tunnel excavation support

- fyk ≥ 550 MPa
- Type: Minova MAI SDA R51L or similar

b. FOREPOLING TUBES

- Steel grade S355
- External diameter 114.3 mm, thickness 16 mm

5.1.1.3 Fiberglass anchors

- Ftk ≥ 800 kN
- Drilling diameter ≥ 100mm
- Type: tubes 60 mm (external diameter) / 40 mm (internal diameter) or similar

5.1.1.4 Waterproofing

The waterproofing system will be considered in the calculation of the final lining. According to our experience into similar projects, waterproofing layer can be modelled by means of elastic springs, having the following stiffness: k_{wtpf} = 60000 KN/m^3

5.1.2 Tunnel technical Buildings (if applicable)

5.1.2.1 Concrete

a. Concrete for foundations

- Concrete class, C25/30
- Exposure classes XC2
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 30 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4

- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

b. Concrete for external walls

- Concrete class, C30/37
- Exposure classes XC4, XD1 and XF1
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 37 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

c. Concrete for internal walls and slab

- Concrete class, C30/37
- Exposure classes XC3
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 37 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm

d. Concrete for top slab

- Concrete class, C30/37
- Exposure classes XC4, XF3
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 37 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

e. Lean Concrete

- Concrete class, C12/15
- Cubic compressive strength Rck ≥ 15 MPa
- Min. cement content: 150kg/m^3

5.1.2.2 Steel reinforcement

Steel ribbed bars for reinforcement:

- Bar diameter, D = according to calculations (between 10 mm and 22 mm)

- Steel grade B500 B
- Yield Strength, fyk ≥ 500 MPa
- Tensile Strength, ftk ≥ 550 MPa
- Minimum value k, ftk/ fyk ≥ 1.08
- Elastic Modulus, E, 210000 MPa

5.1.3 Rockshed (if applicable)

5.1.3.1 Concrete

a. Concrete for foundations

- Concrete class, C25/30
- Exposure classes XC2
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 30 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

b. Concrete for walls

- Concrete class, C30/37
- Exposure classes XC4, XD1 and XF1
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 37 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

c. Concrete for top slab

- Concrete class, C30/37
- Exposure classes XC4, XF3
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 37 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

Lean Concrete

- Concrete class, C12/15
- Cubic compressive strength Rck ≥ 15 MPa
- Min. cement content: 150kg/m^3

5.1.3.2 Steel reinforcement

Steel ribbed bars for reinforcement:

- Bar diameter, D = according to calculations (between 10 mm and 28 mm)
- Steel grade B500 B
- Yield Strength, fyk ≥ 500 MPa
- Tensile Strength, ftk ≥ 550 MPa
- Minimum value k, ftk/ fyk ≥ 1.08
- Elastic Modulus, E, 210000 MPa

5.1.3.3 Cushion layer

Lightweight Expanded Clay Aggregate type Leca 0-30mm or similar, placed in stratum 50cm high, compacted by vibrating plate.

- Specific weight of the supplied material = 390 kg/m^3
- Design specific weight = 6 kN/m^3
- Grains resistance to crushing ≥ 1.5 N/mm^2
- Water absorption after 24 hours < 25%
- Design Young Modulus: E = 25000 kPa
- Friction angle: Φk= 40°
- Max granulometry dimension: Dmax= 30 mm

5.1.4 Culverts

5.1.4.1 Concrete

a. Concrete for foundations

- Concrete class, C25/30
- Exposure classes XC2
- Maximum aggregate diameter, 32 mm
- Cubic Characteristic strength Rck ≥ 30 MPa
- Cement Type: III-IV (EN 197-1)
- Slump Class: S4
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

b. Lean Concrete

- Concrete class, C12/15

- Cubic compressive strength Rck ≥ 15 MPa
- Min. cement content: 150kg/m^3

5.1.4.2 Steel

a. Steel for reinforcement – Ribbed Bars:

- Bar diameter, D = according to calculations (between 14 mm and 22 mm)
- Steel grade B500 B
- Yield Strength, fyk ≥ 500 MPa

5.1.5 Retaining Walls

5.1.5.1 Concrete

- Concrete class, C30/37
- Exposure classes XC4, XD1 and XF1
- Maximum aggregate diameter, 32 mm
- fck = 30 MPa
- Concrete cover = 50mm
- Alkali Aggregate Reaction (AAR) resistant

5.1.5.2 Steel for reinforcement

- Bar diameter, D = according to calculations
- Steel grade B500 B
- Yield Strength, fyk = 500 MPa

5.1.5.3 Rockbolts

a. CEMENTED BOLTS FOR ANCHORS

- Type: Permanent Gewi diam. 25 mm (steel grade B500B) with preinjected corrugated sheating or equivalent
- Tensile Strength, Ftk ≥ 285 kN
- Yield Strength, Fyk ≥ 246 kN
- Drilling diameter ≥ 110 mm

5.1.5.4 Micropiles

b. CEMENTED BOLTS FOR MICROPILES

- Type: Permanent Gewi diam. 32 mm (steel grade B500B) with preinjected corrugated sheating or equivalent
- Tensile Strength Ftk ≥ 466 kN
- Yield Strength, Fyk ≥ 402 kN
- Drilling diameter ≥ 110 mm

c. CEMENTED BOLTS FOR MICROPILES

- Type: Permanent Gewi diam. 40 mm (steel grade B500B) with preinjected corrugated sheating or equivalent

- Tensile Strength Ftk ≥ 728 kN
- Yield Strength, Fyk ≥ 628 kN
- Drilling diameter ≥ 110 mm

5.1.6 Slope Stabilisation with Shotcrete + anchor bolts

5.1.6.1 Shotcrete (Permanent)

- Reference Code: UNI EN 1992 1-1, EN 206-1
- Concrete Class: C30/37
- Characteristic Cubic Strength: Rck >37 MPa
- Cement Type: III-IV (UNI EN 197-1)
- Exposure Class: XC4/XD1/XF1
- Resistant at AAR
- Slump Class: S4
- Maximum Aggregate: 8 mm (EN 12620)
- Steel fiber (where applicable) : minimum 50kg/cum

5.1.6.2 Steel for reinforcement

<u>a. STEEL WIRE MESH</u>

- Bar diameter, D = 6mm
- Mesh spacing = (100x100)mm
- Steel grade min. B500 A
- Yield Strength fyk = 500 MPa

5.1.6.3 Steel for bolts

<u>a. ROCKBOLTS:</u>

- Type: Permanent Gewi diam. 25 mm (steel grade B500B) with preinjected corrugated sheating or equivalent
- Tensile Strength Ftk ≥ 357 kN
- Yield Strength Fyk ≥ 308 kN
- Drilling diameter ≥ 110 mm

5.1.7 Summary of materials

5.1.7.1 Cementitious Materials

Table 5.1 : Lean and Structural Concrete

LEAN AND STRUCTURAL CONCRETE:

	CONCRETE STRENGTH CLASS	MINIMUM CHARACTERISTIC CUBIC STRENGTH R_{ck} (MPa)	EXPOSURE CLASS	MAXIMUM NOMINAL AGGREGATE SIZE (mm)	CONSIST CLAS
LEAN CONCRETE	C12/15 (M15/20mm)	15	X0	20	S2
TUNNEL LINING	C30/37 (M40/32mm)	37	XC4, XD1, XF1	32	S4
ROCKSHED AND TECHNICAL BUILDINGS (FOUNDATIONS)	C25/30 (M30/32mm)	30	XC2	32	S4
ROCKSHED AND TECHNICAL BUILDINGS (EXTERNAL WALLS)	C30/37 (M40/32mm)	37	XC4, XD1, XF1	32	S4
ROCKSHED AND TECHNICAL BUILDINGS (TOP SLAB)	C30/37 (M40/32mm)	37	XC4, XF3	32	S4
TECHNICAL BUILDINGS (INNER WALLS AND SLABS)	C30/37 (M40/32mm)	37	XC3	32	S4
BURIED CULVERTS	C25/30 (M30/32mm)	30	XC2	32	S4

Table 5.2 : Shotcrete

SHOTCRETE:

	STRENGTH CLASS	MINIMUM CHARACTERISTIC CUBIC STRENGTH R_{ck} (MPa)	EXPOSURE CLASS	MAXIMUM NOMINAL AGGREGATE SIZE (mm)	CONSIST CLAS
SHOTCRETE FOR TUNNEL (INTERNAL AREAS)	C25/30 (M30/8mm)	30	XC4, XD3	8	S4
SHOTCRETE FOR ROCK					

Table 5.3: Cement-based grout for rock bolts/anchor

CEMENT-BASED GROUT FOR ROCK BOLTS/ ANCHORS INJECTION

TYPE:	HIGH STRENGTH, NON-SHRINK CEM CONSTRUCTION GROUT WITH AGGF SUITABLE FOR THE EFFECTIVE POU
W/C:	0.5
PLASTICIZER:	1% - 2% ON WEIGHT

5.1.7.2 Steel Materials

Table 5.4 : Steel for reinforcement

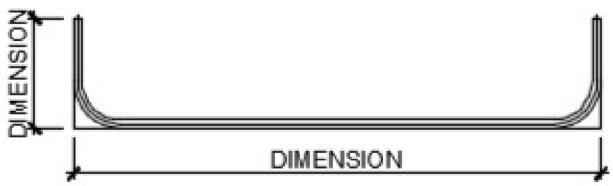
STEEL REINFORCEMENT

1. MIN. COVER REINFORCEMENT = 50mm (TYPICAL), UNLESS NOTED OTHERWISE

2. MIN. OVERLAPPING: 50 Ø, UNLESS NOTED OTHERWISE

3. PROPERTIES:

TYPE (EN 10080):	RIBBED BARS
CHARACTERISTIC YIELD STRENGTH f_{yk}:	≥ 500 MPa
TENSILE STRENGTH f_{tk}:	550 MPa
MINIMUM VALUE $k = f_{tk} / f_{yk}$:	≥ 1.08
CHARACTERISTIC STRAIN ε_{uk}:	≥ 5% (AT MAXIMUM FORCE)
REFERENCE CODE:	EN 1992 1-1, TAB. C.1.



LENGTHS ARE MEASURED OUT-TO-OUT OF BARS

Ø mm	Ri mm	a mm
10	25	47
12	30	57
14	35	66
16	40	75
18	72	127
20	80	141
22	88	156
24	96	170

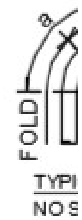


Table 5.5: Structural Steel

STRUCTURAL STEEL FOR STEEL RIBS AND LATTICE GIRDERS

STRUCTURAL STEEL	PROFILE	TYPE	STEEL GRADE
STEEL RIBS	HEB 180	-	S355
LATTICE GIRDER	ACCORDING TO TUNNEL	JB-3-GURT OR	S235/S500

Table 5.6: Temporary Rockbolts for tunnel excavation

ROCKBOLTS

	SWELLEX PM 16	SWELLEX PM 24	BELLOLI BELCEM M33 OR SIMILAR	BELLOLI BELC M27 OR SIMIL
DIAMETER	ø 16 mm	ø 24 mm	ø 33 mm	ø 27 mm
TENS. STRENGTH, f_{tk}	≥ 160 kN	≥ 240 kN	≥ 460 kN	≥ 350 kN
YIELD STRENGTH, f_{yk}	≥ 130 kN	≥ 200 kN	≥ 364 kN	≥ 280 kN
DRILLING DIAMETER	≥ 50 mm	≥ 50 mm	≥ 60 mm	≥ 60 mm

Table 5.7 : Permanent rock bolts for slope stability

ROCK BOLTS

	GEWI BAR 32 OR SIMILAR	GEWI PLUS
NOMINAL DIAMETER Ø	32 mm	43
CROSS SECTIONAL AREA	804 mm^2	14
GRADE	S500/550	S6
YIELD STRENGTH, f_{yk}	≥ 500 MPa	≥ 6
TENSILE STRENGTH, f_{tk}	≥ 550 MPa	≥ 8
LOAD AT YIELD	402 kN	9
ULTIMATE LOAD	442 kN	1
WEIGHT	6.31 kg/m	11
OTHER REQUIREMENTS	CORRUGATED SHEATING	CORRUGATED SHI

Table 5.8 : Temporary Pre-tensioned anchor for excavation stability at portals

PRE-TENSIONED GROUTED ANCHOR

HARMONIC STEEL	0.6"
GRADE	S1670/1860
YIELD STRENGTH, f_{yk}	1670 MPa
TENSILE STRENGTH, f_{tk}	1860 MPa
NUMBER OF STRANDS	5
NOMINAL STRAND AREA, Ai	140 mm^2
LOAD AT YIELD	234 kN/STRAND
ULTIMATE LOAD	260 kN/STRAND

Table 5.9: Forepoling

FOREPOLING BARS

F_{yk}:	≥ 550 MPa
TYPE:	MINOVA MAI SDA R51L OR SIMILAR
LENGTH:	3 m
SPACING (TRANSVERSAL):	0.375 m

FOREPOLING TUBES

STEEL GRADE: >	S355
EXTERNAL DIAMETER:	114.3 mm, THICKNESS 16 mm

5.1.7.3 Fiberglass

Table5.10 : Fiberglass

FIBER GLASS ANCHORS

F_{tk}:	≥ 800 kN
DRILLING DIAMETER:	≥ 100 mm
TYPE:	TUBES 60 mm (EXTERNAL DIAMETER) / 40 mm (INTERNA

5.2 Design Working Life

Design Working Life of structures is defined as "the period of time during which the construction is assumed to be used for intended purposes, with anticipated maintenance, but without substantial repairs".

According to EN 1990 (Eurocode 0) the design working lives (Tunnel and Rock Shed) of Category 5 – Monumental building structures, bridges and other civil engineering structures – 100 years.

According to EN 1990 (Eurocode 0) the design working lives (Technical Building and Other Road structures,Slope stabilization and mitigation related) of Category 4 – Building structures and other common structures – 50 years.

According to EN 1990 (Eurocode 0) the design working lives of civil engineering structures with low or moderate economic repercussio

This requirement has to be met by means of a design that includes an appropriate selection of the structural solutions and construction materials, a careful construction compliant with the design and a suitable inspection of the design, where appropriate. Construction and operation activities, together with an appropriate use and maintenance of the structures are as well of major importance to guarantee such requirement.

For temporary structures, such as for the excavation works at the tunnel portals, a design working life of 10 years has been considered.

Table 5.11 Design Working Life (EN 1990)

Elements	Design working life category	Design working life (years)
Tunnel and Rock Shed related structures	5	100
(Technical Building and Other Road structures,Slope stabilization and mitigation related)	4	50

5.3 Design Approach

Structural and geotechnical design is carried out according to Eurocodes, with reference to the semi-probabilistic method (limit state). Load Combinations, safety and partial factor coefficients, design and verifications are considered both at the Ultimate Limit State (USL) and at the Serviceability Limit State (SLS), according to Eurocodes.

5.3.1 Design conditions

The relevant design conditions are considered taking into account the circumstances under which the structure is requested to fulfil its function, which can be classified as:

- transient situations, with reference to temporary conditions applicable to a structure, e.g. an executive phase or repair;
- persistent situations, which refer to normal service conditions;

- accidental or exceptional situations, which refer to exceptional conditions applicable to a structure, ex. rockfall, collisions, fires, explosions, etc..;
- seismic situations, which refer to conditions applicable to the structure when subjected to seismic events.

Table 5.12 Design Conditions

Design conditions	Description
Persistent	Condition of normal use
Transient	Temporary conditions, e.g. during execution or repair
Accidental	Exceptional conditions structure/exposure, e.g. fire, explosion
Seismic	Condition when structure subject to seismic events

Note that each category requires a specific analysis of the time horizon in which it occurs; the design situations can occur throughout the design life of a structure or be limited to a specific executive phase or repair.

5.3.2 Principles of Limit State Design

5.3.2.1 Ultimate Limit State (ULS) – Structure design

The Ultimate Limit States (ULS) for structural design concern:

- The safety of people, and/or
- The safety of the structure.

The following ultimate limit states are relevant for structures design (EN 1990):

- **EQU**: loss of equilibrium of structure, considered as rigid body;
- **STR:** internal failure or excessive deformation of the structure or structural members,

When considering a limit state of static equilibrium of the structure (EQU), it shall be verified that:

Ed,dst ≤ Ed,stb

where:

Ed,dst is the design value of the effect of destabilising actions ; Ed,stb is the design value of the effect of stabilising actions.

When considering a limit state of rupture or excessive deformation of a section, member or connection (STR), it shall be verified that:

Ed ≤ Rd

where:

Ed is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments;

Rd is the design value of the corresponding resistance.

When considering a limit state of rupture or excessive deformation of a section, member or connection (STR), it shall be verified that:

Ed ≤ Rd

where:

Ed is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments;

Rd is the design value of the corresponding resistance.

ULS load combinations considered for design are here briefly summarized.

5.3.2.2 Ultimate Limit State (ULS) – Geotechnical design

When defining the design situations and the limit states, the following factors should be considered:

- site conditions with respect to overall stability and ground movements;
- nature and size of the structure and its elements, including any special requirements such as the design life;
- conditions with regard to its surroundings (e.g.: neighbouring structures, traffic, utilities, vegetation, hazardous chemicals);
- ground conditions;
- ground-water conditions;
- regional seismicity;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

Limit states can occur either in the ground or in the structure or by combined failure in the structure and the ground.

The following ultimate limit states are relevant for project structures (EN 1997):

- EQU: loss of equilibrium of the ground, considered as a rigid body, in which the strengths of the ground are insignificant in providing resistance.
- GEO: failure or excessive deformation of the ground where the strengths of soil are significant in providing resistance.

When considering a limit state of static equilibrium or of overall displacements of the structure or ground (**EQU**), it shall be verified that:

Ed,dst ≤ Ed,stb + Td

where:

Ed,dst is the design value of the effect of destabilising actions ;

Ed,stb is the design value of the effect of stabilising actions;

Td is design value of total shearing resistance that develops around a block of ground in which a ground of tension piles is placed, or on the part of the structure in contact with the ground.

When considering a limit state of rupture or excessive deformation of a section, member or connection (**GEO**), it shall be verified that:

Ed ≤ Rd

where:

Ed is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments;

Rd is the design value of the corresponding resistance.

According to EN 1997-1 ultimate state limit design for STR and GEO the manner in which equations of "design resistance" and "design effects on action" are applied shall be determined using one of three Design Approaches.

Design Approach 1 has been considered in the project and it is detailed in the following.

5.3.2.2.1 Design Approach 1

According to Eurocode EN 1990, design of structural members (STR) involving geotechnical actions and the resistance of ground (GEO) should be verified using only one of the different three approaches supplemented for geotechnical actions and resistances, by EN 1997.

For the structural verifications in this DPR, Approach 1 has been adopted, where partial factors are applied to actions and to ground strength parameters.

Except for the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: A1 "+" M1 "+" R1

Combination 2: A2 "+" M2 "+" R1

where "+" implies: "to be combined with".

For the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 2: A2 "+" M2 "+" R4

In Combination 1, partial factors are applied to actions and to ground strength parameters. In Combination 2, partial factors are applied to actions, to ground resistances and sometimes to ground strength parameters.

In Combination 2, set M1 is used for calculating resistances of piles or anchors and set M2 for calculating unfavourable actions on piles owing e.g. to negative skin friction or transverse loading.

If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. However, different combinations may be critical to different aspects of the same design..

5.3.2.3 Serviceability Limit State (ULS) – Structure design

The Serviceability Limit State (SLS) concerns:

- the functioning of the structure or structural members under normal use;
- the comfort of people;
- the appearance of the construction works.

The verification of serviceability limit states is based on criteria concerning the following aspects:

a) deformations that affect

- the appearance,
- the comfort of users, or
- the functioning of the structure, or that cause damage to finishes or non-structural members;

b) damage that is likely to adversely affect

- the appearance,
- the durability, or
- the functioning of the structure.

The combination of actions to be taken into account in the relevant design situation should be appropriate for the serviceability requirements and performance criteria being verified:

Ed ≤ Cd

Ed design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination.

Cd limiting design value of the relevant serviceability criterion: stress limitation, crack control and deformation control.

5.3.2.4 Serviceability Limit State (ULS) – Geotechnical

Verification for serviceability limit states in the ground or in a structural section, element or connection, shall either require that:

Ed ≤ Cd

Ed design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination.

Cd limiting design value of the relevant serviceability criterion: stress limitation, crack control and deformation control.

A limiting value for a particular deformation is the value at which a serviceability limit state is deemed to occur in the supported structure. For retaining structures, such as wall as according to Swiss standard,

5.3.3 Combination of Actions

For each critical load case, the design value of the effect of actions shall be determined by combining the values of actions are considered to occur simultaneously. Each combination of actions should include a leading variable action, or an accidental action.

The combination of actions for ULS are defined symbolically by the following expressions.

5.3.3.1 ULS fundamental static load combination

$$E_d = E\left\{\gamma_{G,j} G_{k,j} \; ; \gamma_P P \; ; \gamma_{Q,1} Q_{k,1} \; ; \gamma_{Q,i} \psi_{0,i} Q_{k,i}\right\} \quad j$$

$$\sum \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

where:

x partial factor taking account of possible unfavourable deviations of the action from the representative values of the actions

Ψ partial factor depending on the limit state considered.

$\Psi_0 Q_k$ combination value, chosen so that the probability the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action

G_k characteristic value of permanent action

Q_k characteristic value of variable action

P relevant representative value of a prestressing action = 0

5.3.3.2 ULS accidental load combination

$$E_d = E\{G_{k,j} ; P ; A_d ; (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} ; \psi_{2,i}Q_{k,i}\}$$

$$\sum G_{k,j} \text{"+"} P \text{"+"} A_d \text{"+"} (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} \text{"+"} \sum \psi_{2,i}($$

where:

γ partial factor depending on the LS considered;

$\Psi_1 Q_k$ frequent value of the variable action to be considered

$\Psi_2 Q_k$ quasi-permanent value of the variable action to be considered

G_k characteristic value of permanent action;

Q_k characteristic value of variable action;

P relevant representative value of a prestressing action = 0;

Ad design value of the accidental action considered (*rockfall*)

5.3.3.3 ULS seismic load combination

$$E_d = E\{G_{k,j} ; P ; A_{Ed} ; \psi_{2,i}Q_{k,i}\} \quad j \geq$$

$$\sum G_{k,j} \text{"+"} P \text{"+"} A_{Ed} \text{"+"} \sum \psi_{2,i}Q_{k,i}$$

where:

Ψ partial factor depending on the LS considered;

$\Psi_2 Q_k$ quasi-permanent value, chosen so that the total period of time for which it will be exceeded is a large fraction of the reference period;

G_k characteristic value of permanent action;

Q_k characteristic value of variable action;

P relevant representative value of a prestressing action = 0;

AEd design value of the seismic action.

5.3.3.4 Serviceability Limit states

The combinations of actions to be taken into account in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified.

Characteristic combination (adopted for irreversible limit state)

$$\sum_{i>1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1} \text{"+"} \sum_{i>1} \psi$$

Frequent combination (adopted for reversible limit state)

$$\sum_{i\geq 1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi$$

Quasi-permanent combination (adopted for long-term effects and appearance of the structure)

$$\sum_{i>1} G_{k,j} \text{"+"} P \text{"+"} \sum_{i>1} \psi_2$$

5.3.4 Partial Safety Factors

5.3.4.1 Ultimate Limit States Partial Factors

The partial factors are briefly summarized in the following tables. For further details and specific situations, see the relevant reports.

Table 5.13 *STR/GEO ULS static condition coefficients*

		Coefficient γ_F	EQU	STR/GEO A1	STR/GEO A2
Permanent load (structures)	Favorable	γ_{G1}	0.90	1.00	1.00
	Unfavorable		1.10	1.35	1.00
Permanent load (not structures) (defined)	Favorable	$\gamma_{G2\,(A)}$	0.90	1.00	1.00
	Unfavorable		1.10	1.35	1.00
Live load: wind, snow, others	Favorable	γ_{Qi}	0.00	0.00	0.00
	Unfavorable		1.50	1.50	1.30
Shrinkage, creep, settlements	Favorable	$\gamma_{\varepsilon 2}, \gamma_{\varepsilon 3}, \gamma_{\varepsilon 4}$	0.00	0.00	0.00
	Unfavorable		1.20	1.20	1.00

Table 5.14 For traffic load, where relevant, the following coefficients have been considered.

		Coefficient, γ_F	EQU	STR/GEO A1	STR/GEO A2
Live load: Traffic load	Favorable	$\gamma_{Q(TS)}$	0.00	0.00	0.00

(if relevant, where applied)	Unfavorable		1.35	1.50	1.15

According to EN 1997-1 the same partial factors on actions are applied in geotechnical structures design for verification of equilibrium limit sate (EQU).

The partial factors for actions for the ultimate limit states *in the accidental and seismic design situations* have been assumed equal to 1.0, according to Eurocode EN 1990.

Table 5.15 Partial coefficient for nominal soil/rock nominal parameters (M)

Material	***Normal Strength***		***EQU***	***STR/GEO M1***	***STR/GEO/M2***
Ground	***Friction Angle tanΦ'_k***	$\gamma_{\Phi'}$	***1.25***	***1.00***	***1.25***
	Cohesion c'_k	$\gamma_{c'}$	***1.25***	***1.00***	***1.25***
	Undraind Strength c_{uk}	γ_{cu}	***1.40***	***1.00***	***1.40***
	Unit Self Weight, γ_T	γ	***1.00***	***1.00***	***1.00***

For the ground partial factors, for the ultimate limit states *in seismic design situations*, M2 values are adopted, according to EN 1998-5, par. 3.

Table 16 Partial coefficient for nominal material strengths (M) (EN 1992-1-1)

Material	Nominal strength		Transient/ Permanent	Seismic/ Accidental
Concrete	Compression/tension/ shear strength f_{ck}, f_{ctk} , v_k	γ_c	1.50	1.20
Steel reinforcement	Yield strength f_{yk}	γ_s	1.15	1.0

Table 17 Partial coefficient for global strengths (R)

Element	**Strength**	**Symbol**	**R1**	**R4**
Shallow foundation (EN 1997, A.5)	Bearing capacity Sliding	γ_R γ_R	1.00 1.00	- -
Piles foundation (EN 1997, A.6)	Base Bearing capacity Lateral Bearing capacity (compr.) Total bearing capacity (compr.)	$\gamma_{R,b}$ $\gamma_{R,s}$ $\gamma_{R,s}$	1.00 1.00 1.00	1.3 1.3 1.3
	Lateral Bearing capacity (tensile)	$\gamma_{R,s,t}$	1.25	1.6
Pre-tensioned Anchoring (EN 1997, A.12)	Transient	$\gamma_{R,a,t}$	1.1	1.1
	Permanent	$\gamma_{R,a,p}$	1.1	1.1
Retaining structures (EN 1997, A.13)	Bearing capacity	$\gamma_{R,V}$	1.00	-
	Sliding	$\gamma_{R,H}$	1.00	-
	Downstream Earth Strength	$\gamma_{R,E}$	1.00	-

Global stability (EN 1997, A.14)	Earth Resistance	$\gamma_{R,E}$	1.00	-

For verifications of structural (STR) and geotechnical (GEO) limit states of pile foundations, the following correlation factors ξ shall be applied to derive the characteristic resistance of axially loaded piles:

- ξ3 on the mean values of the calculated resistances from ground test results;
- ξ4 on the minimum value of the calculated resistances from ground test results;

Correlation factors ξ3, ξ4, to derive characteristic values from ground test results (n - the number of profiles of tests).

Table 18 Correlation factors for piles

ξ for n =	**1**	**2**	**3**	**4**	**5**	**7**	**10**
ξ3	1.40	1.35	1.33	1.31	1.29	1.27	1.25
ξ4	1.40	1.27	1.23	1.20	1.15	1.12	1.08

5.3.4.2 Serviceability Partial Factors

Table 19 SLS actions partial factors

Category	Actions	Description	Ψ_0	Ψ_1	Ψ_2
B	Imposed loads in buildings	office areas	0.70	0.50	0.30
E	Imposed loads in buildings	storage areas	1.00	0.90	0.80
G	Traffic area, 30kN < vehicle weight ≤160 kN	roads on surface	0.70	0.50	0.30
-	Snow loads on buildings		0.70	0.50	0.20
-	Wind loads on buildings		0.60	0.20	0.00
-	Temperature (non-fire) in buildings		0.60	0.50	0.00
	Actions (construction)	Description	Ψ_0	Ψ_1	Ψ_2
	Construction loads	Qc	1.00	-	1.00

Ψ- values are valid also for seismic and accidental conditions, but considering that during accidental events other live loads such as snow and wind have not to be considered as simultaneously with the accidental action.

5.3.5 Durability

The structure is designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance.

In order to achieve an adequately durable structure, the following should be taken into account:

- the intended or foreseeable use of the structure;
- the required design criteria;
- the expected environmental conditions;
- the composition, properties and performance of the materials and products;
- the properties of the soil;
- the choice of the structural system;
- the shape of members and the structural detailing;
- the quality of workmanship, and the level of control;
- the particular protective measures;
- the intended maintenance during the design working life.

The environmental conditions is identified at this design stage, so that their significance can be assessed in relation to durability and adequate provisions can be made for protection of the materials used in the structure.

5.3.5.1 Environmental exposure classes

According to standards EN 206-1, EN 1992-1-1, the following classes of environmental exposure and minimum strength class of concrete have been considered for the different structural elements of the main protective structures, as reported in the following table. The maximum water-cement ratio and minimum cement or combination content are given referring to normal-weight concrete with 20 mm maximum aggregate size. For further details on materials see specific report and drawing.

Table 20 Exposure classes and concrete strength classes for durability according to EN 206

	Exposure Class	Min. Concrete strength class	Max Water/Cement Ratio	Min Cement Content (kg/cum)
Rockshed				
Foundations	XC2	C25/30	0.60	280
Walls and Columns	XC4, XD1, XF1	C30/37	0.50	300
Top Slab	XC4, XF3	C30/37	0.50	320
Tunnel				
Shotcrete (temporary)	XC2	C25/30	0.60	280
Inner lining	XC4, XD1, XF1	C30/37	0.50	300
Excavation supports				
Shotcrete (permanent)	XC4, XD1, XF1	C30/37	0.50	300
Retaining Walls				
Concrete	XC4, XD1, XF1	C30/37	0.50	300
Tunnel technical building				
Foundations	XC2	C25/30	0.60	280

Perimetral Walls	XC4, XD1, XF1	C30/37	0.50	300
Top Slab	XC4, XF3	C30/37	0.50	320
Internal Slab/Walls	XC3	C30/37	0.50	300

5.3.5.2 Concrete cover

The concrete cover equal to the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups) and the nearest concrete surface.

Nominal cover $c_{nom} = c_{min} + \Delta c_{dev}$

where:

Δc_{dev} additional cover for tolerances (= 10mm)

cmin minimum cover = max[$c_{min,b}$; $c_{min,dur}+\Delta c_{dur,\gamma}-\Delta c_{dur,st}-\Delta c_{dur,add}$;10mm]

cmin,b minimum cover due to bond requirements: reinforcement steel -> separated rebars:

$c_{min,b}$ = diameter of bar ($c_{min,b,max}$ = 28mm)

$c_{min,dur}$ minimum cover due to environmental conditions ($c_{min,dur}$ = 35mm for XD1)

$\Delta c_{dur,\gamma}$ additive safety element = 5mm

$\Delta c_{dur,st}$,stainless steel (= 0)

$\Delta c_{dur,add}$,additional protection (e.g. coating) (= 0)

Concrete cover value is given for reinforcement steel just for durability assessment.

The following concrete covers for main reinforcement have been adopted, according to the considered exposure classes and providing an unified specification for all structural elements:

50 mm for all formed concrete exposed to weather or in contact with the ground or weather.

50 mm for concrete slabs and walls and for bars and girders neither exposed to weather nor in contact with the ground.

5.3.6 Design verifications

Ultimate limit states (ULS) and serviceability limit state (SLS) verifications have been considered according to Eurocodes.

Safety checks are performed by means of the Partial Factors Method: for all relevant design situations, verifying if the limit states are not exceeded when design values of the actions, material properties and geometric dates are introduced in structural and load models.

The non-exceedance limit check allows to state that the probability of reaching a certain limit situation is less than the value set by the standard during the structure life or during a timeframe of reference in case of execution/constructive phase.

The verifications ensure that:

- Assumed design actions do not cause the collapse of the structure or the ground (even in exceptional/accidental situations);
- Effects of the assumed design actions do not exceed the design strength of the structure at the ultimate limit state;
- Effects of the assumed actions do not exceed the functionality criteria for the service limit state.

Ultimate limit states connected both to the structure collapse (or the whole structure-ground) and to a partial failure are considered; Service ultimate states that match the conditions beyond which specific operational requirements connected to the structure or its elements are no longer met are taken into account. For geotechnical verifications refer also to 5.6.

5.3.6.1 Performance Level of structures

ULS

For the design of new structures these performances are considered at ULS for permanent/transient loads conditions

Table 21 ULS permanent/transient structures performance.

Behaviour	Performance
Non-linear materials/sections. Linear/Non-linear structure behaviour.	Safety, no collapse: $E_d \leq R_d$

For the design of new structures these performances are considered at ULS for accidental loads conditions:

Table 22 ULS accidental structures performance.

Behaviour	Performance
Non-linear materials/sections.	Safety, no collapse: $E_d \leq R_d$
Linear/Non-linear structure behaviour.	Fire, REI

SEISMIC PERFORMANCE

According to EN 1998 to the design and construction of civil engineering works in seismic regions, structures and geotechnical structures of new structures should ensure that in the event of earthquakes:

- human lives are protected;
- damage is limited;
- structures important for civil protection remain operational.

5.3.6.2 SLS verifications

5.3.6.2.1 SLS Stress limits for concrete structures

According to EN 1992-1 the compressive stress in the concrete is limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could result in unacceptable effects on the function of the structure.

Characteristic load combination: $\sigma_c \leq k_1 \times fck = 0.60 \times f_{ck}$

Quasi-permanent load combination: $\sigma_c \leq k_2 \times fck = 0.45 \times f_{ck}$

According to EN 1992-1 the tensile stresses in the reinforcement are limited in order to avoid inelastic strain, unacceptable cracking or deformation.

Characteristic load combination: $\sigma_s \leq k_3 \times f_{yk} = 0.8 \times f_{yk}$

Imposed deformations: $\sigma_s \leq k_4 \times f_{yk} = 1.0 \times f_{yk}$

5.3.6.2.2 SLS Crack control for concrete structures

A limiting calculated crack width, w_{max}, taking into account the function and nature of the structure and the costs of limiting cracking, is established according to EN 1992-1-1 (Table 7.1N), for the different exposure classes.

For steel reinforced concrete, suggested values are reported.

Table 23 Concrete crack control

Exposure class	Reinforced members Quasi-permanent load combination
XC2 –XC4 - XD1	W_{max} = 0.3 mm

For temporary structures, that develops loads for short time of construction, crack control will be neglected.

Measures to control leakage such as drains and water stops shall be used.

5.3.6.2.3 Deflection control

According to Eurocode EN 1992-1-1, deflections of slabs and beams should satisfy the following limits to avoid irreparable damages of the element and of adjacent parts of the structure, respectively:

$\delta \leq L/250$, for quasi-permanent load combinations during constructions;

$\delta \leq L/500$, for quasi-permanent load combinations after constructions (live load).

The appearance and general utility of the structure could be impaired when the sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250. The sag is assessed relative to the supports.

Deflections that could damage adjacent parts of the structure should be limited. For the deflection after construction, span/500 is normally an appropriate limit for quasi-permanent and frequent loads. Other limits may be considered, depending on the sensitivity of adjacent parts.

5.4 Design Load

Design loads for all structures will be determined in accordance with the criteria described below, unless the applicable building code requirements are more stringent.

5.4.1 Tunnel

5.4.1.1 Concrete dead load

Characteristic value of the shotcrete self-weight is: 25 KN/m^3

Characteristic value of the cast in situ concrete is: : 25 KN/m^3

5.4.1.2 Unstable blocks

The weight of the rock shall be assumed for all lithologies according to the geological/geotechnical evaluations.Moreover, parameters for joints resistance shall be assumed in the calculations, based on performed geomechanical characterization.

5.4.1.3 Rock loosening and soil pressure

Loosening of the rock mass around the tunnel shall be estimated through well recognized design approaches as a parabolic-linear load, according to Terzaghi's theory) or derived by FEM calculations. Loosening heights and the resulting maximum loads (at the tunnel key) both on primary lining and on inner lining must be defined according to geomechanical characteristics estimated along the tunnel.

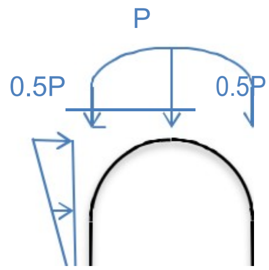


Figure 5.1: Example of loosening load at the tunnel contour

5.4.1.4 Seismic load

Earthquake is not considered for the underground part of the tunnel, as no shear lines (faults) has been detected along the tunnel.

5.4.2 Tunnel Technical Building

5.4.2.1 Dead load

Characteristic value of the cast in situ concrete is:

Characteristic value of the shotcrete self-weight is: 25 KN/m^3

Characteristic value of the cast in situ concrete is: : 25 KN/m^3

Moreover, loads from permanent equipments and installations shall be considered according to the final design to be provided by the Contactor.

5.4.2.2 Overburden

The following minimum self weight of the filling ground shall be assumed: 2 KN/m^3

Overburden loads follow the geometry of the filling ground according to the project.

5.4.2.3 Earth thrust

Earth pressure at rest is calculated based on the filling ground parameters, by means of the Jaky's formula:

$K_0 = 1 - \sin(\Phi_d)$

where Φ_d = design angle of friction.

5.4.2.4 Live load

On the buildings slabs, a minimum distributed live load equal to 3 kN/m^2 has to be assumed. Moreover, live loads according to technical specifications of the equipment designed by the Contractor shall be considered.

5.4.2.5 Seismic load

Earthquake horizontal and vertical acceleration shall be considered, based on a peak value of ground horizontal acceleration for reference period of 475 years for the project area a_g ***[Specify as per project area, ex. 0.31g].***

5.4.3 Cut & Cover tunnel

5.4.3.1 Dead load

Characteristic value of the cast in situ concrete is: 25 KN/m^3

5.4.3.2 Overburden

The following minimum self weight of the soil shall be assumed: 21 KN/m^3

Overburden loads follow the geometry of the filling ground according to the project.

5.4.3.3 Live load

The following surface live load shall be considered in the cut & cover tunnel stretch: 5 KN/m^3

5.4.3.4 Earth thrust

Earth pressure at rest is calculated based on the filling ground parameters, by means of the Jaky's formula:

$K_0 = 1 - \sin(\Phi_d)$

where Φ_d = design angle of friction.

5.4.3.5 Seismic load

Earthquake horizontal and vertical acceleration must be considered.

Pseudo static analyses, with the peak horizontal acceleration expected for a given return period, shall be performed.

The peak value of ground horizontal acceleration for reference period of 475 years for the project area is a_g ***[Specify as per project area, ex. 0.31g].***

According to EC8:

$$k_h = \alpha \frac{S}{r}$$

$$k_v = \pm 0.5 k_h$$

According to provided geological information indicating a ratio between components of the seismic action a_v/a_h=0.67, k_v will be assumed as:

$k_v = 0.7\ k_h$.

The following parameters shall be assumed:

$\alpha = a_g / g$ (i.e. 0.31g/g = 0.31);

S =1.2 (soil factor on rock shall be 1.0. In the present case 1.2 is assumed as a conservative value);

r = 1.0 (rigid structure).

Therefore:

Ex.: Kh = 0.37; Kv = 0.26.

5.4.4 Rockshed

5.4.4.1 Dead Load

Self-Weight

The self-weight of the structural members has been calculated assuming a characteristic unit weight of reinforced concrete of 25 kN/m^3.

Cushion material on top slab

The equivalent distributed loads over the top slab have been determined in relation to the effective design cushion layer thickness "Hl" upon the top slab and assuming a lightweight expanded clay unit weight equal to $\gamma = 6$ kN/m^3 (dry unit weight).

The resulting pressure is given by:

$P=\gamma\ H$

Surcharge on ground

The vertical pressure acting on ground level is evaluated considering the contribution of the Leca cushion and backfill material loading the ground level at the top slab level. Considering the relevant height and

unit weight equal to $\gamma_{Fill} = 6$ kN/m^3 for the Leca and $\gamma_{Fill} = 20$ kN/m^3 for the filling material , the resulting pressure is given by:

$P=\gamma H$

Backfill horizontal earth pressure

Based on the vertical pressure due to the surcharge on ground, earth pressure acting on the mountain wall side has been calculated as hydrostatic pressure acting on the total wall height, according to equation:

$P = \gamma\ H(z)\ k_0$

Where :

γ= backfill unit weight (kN/m^3)

$K_0 = 1 - \sin(\Phi')$ = at rest coefficient of lateral earth pressure

Φ' = internal friction angle of the backfill

H = calculation total height of the rockshed

For design, a selected granular filling, well compacted and placed in stratum 50cm high, with Self weight =20 kN/m^3 and Angle of friction $\Phi \geq 40°$ have been assumed.

5.4.4.2 Live Load

Traffic Load

According to Eurocode surface traffic load due to pedestrian has been considered as an uniform pressure acting on the bottom slab.

Snow Load

According to Nepal weather conditions, different climates according to altitude can be observe. Since according to temperature temperatures during the year except altitude 3000m,are generally greater than 0°C, snow load could been neglected.

Wind Load

Horizontal forces due to the wind action, both inward (towards mountain side) and outward (towards valley side) has been assumed negligible for this kind of structure.

5.4.4.3 Temperature

A temperature variation on the top slab, given by two components:

A uniform temperature component ΔT_U = +/- 15°C

A linearly varying temperature difference component along z-axis, ΔT_{MZ} = +/-5°

shall been considered, assuming a thermal expansion coefficient for concrete equal to $\alpha=1.2E^{-5}$ [1/°C],

5.4.4.4 Shrinkage

The shrinkage effects have been considered according to Standards approaches and a uniform temperature component ΔT, equal to:

$$\Delta T = \varepsilon_{sh} / \alpha$$

has been applied to the concrete top slab, where the thermal expansion coefficient for concrete equal to α = 1.2 E-5 [1/°C], and ε_{sh} is the shrinkage coefficient.

5.4.4.5 Accidental Load

Rockfall Load

According to the rockfall hazard analysis, the following assumptions has to be considered to estimate the rockfall design energy:

- The calculation volume is equal to the characteristic volume ($V_{cal} = V_k$) if the rockfall trajectories do not show significant rebounds;
- if the rockfall trajectories show at least 2 significant rebounds, it can be assumed that the characteristic boulder is shattered into elements equal in size to the average volume (V_m) of the boulders resulting by the static analysis of the recorded volumes by site surveys. This phenomenon is considered plausible considering the degree of interlocking of the rock mass and the conditions of the joints. In this case, the design coulder volume shall be increased by a partial factor equal to 1.5: $V_{cal} = 1.5 \times V_m$

Impact energy has to be determined according to the rockfall hazard analysis but the characteristic impact energy assumed for design should be at least Ec ≥ 15670 kJ (minimum design value). ***[Specify as per project area]*.**

The load estimation has been carried out according to ASTRA Swiss Guidelines, considering a pseudo-static approach based on an equivalent static force Ad that considers the impact load Fk of the rockfall on the rockshed, according to the following Equations:

$$A_d = C \cdot F_k$$

$$F_k = 2.8 \cdot e^{-0.5} \cdot r^{0.7} \cdot M_{E,k}^{0.4} \cdot \tan\varphi_k \cdot \left(\frac{m_k, v_k}{\ldots} \right.$$

where:

- A_d Static equivalent force on design level [kN]
- C Coefficient to account for ductile (C=0.4) or brittle (C=1.2) failure of the structure [-]
- F_k Impact load [kN]
- e Thickness of cushion layer [m]
- r Radius of an equivalent sphere [m]

- $M_{E,k}$ Soil modulus of the cover layer [kN/m^2]
- ϕ_k Internal friction angle of the cover layer [°]
- m_k Characteristic block mass [t]
- v_k Characteristic impact velocity [m/s]

It is worth noting that the application of the guideline is limited to a penetration depth in the cushion layer of smaller than half of the cushion thickness; thus it is evident that cushion layer properties are of great importance to gain the potential enhancement of the loading capacity of existing structures.

This model is based on the following assumptions:

- The concrete structure is sufficiently rigid to neglect, in the elastic range, the influence of structure on the transmitted impact force;
- The concrete slab is sufficiently rigid to allow the impact load redistribution along the transversal and the longitudinal dimensions;
- The horizontal component of impact can be regarded separately;
- The loading surface is plane;
- The soil stiffness is independent from the impact velocity;
- The direct shear failure in soil is not considered;
- The concentrations of the transmitted forces do not exhibit premature punching failure;
- The diameter of the punching cone is determined with the maximum possible penetration, that must be smaller than half of the cushion thickness.

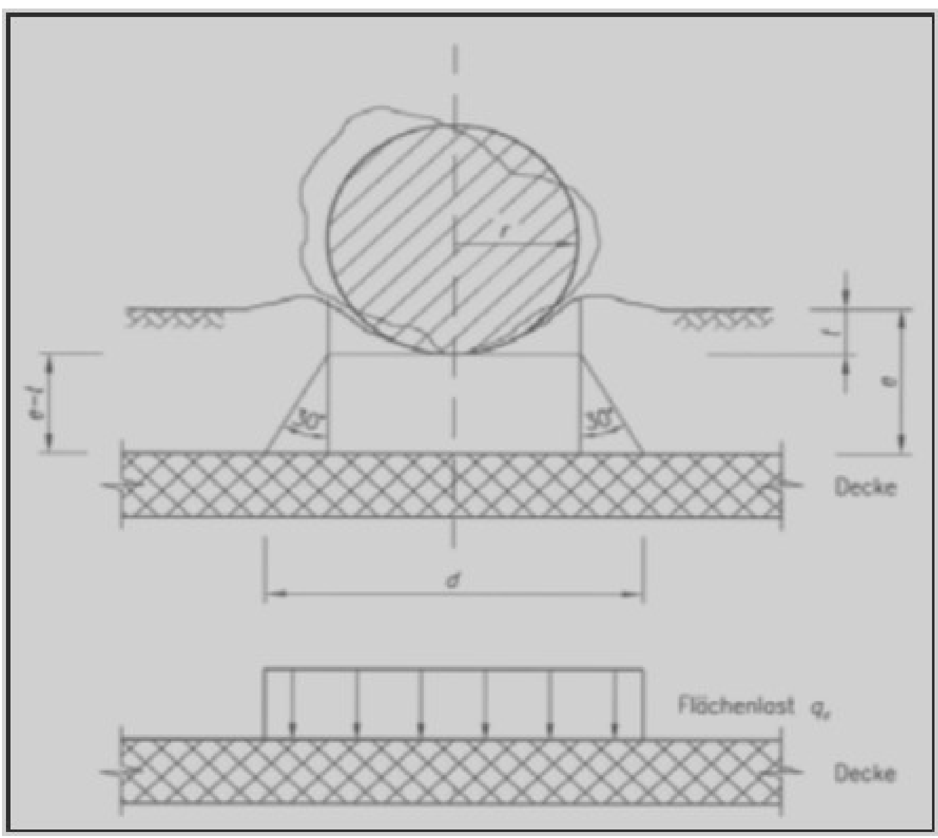


Figure 5.2 - *Load distribution of the equivalent static load according to [ASTRA, 2008]*

A transversal and longitudinal redistribution of the design equivalent static force on a square area given by the rockfall analyses and according to the damping cushion thickness and Swiss guidelines recommendations, has to be considered. The load has to be assumed to distribute with an angle of 45° up to the top slab medium plane.

An horizontal component of the impact has to be considered conservatively as 10% of the vertical load.

In order to evaluate a complete scenario of presumable rockfall impact locations, different location of the rock boulder have to be analysed.

5.4.4.6 Seismic Load

According to EN 1998-1/5, the structure has to be designed to fulfil its function during and after an earthquake, without suffering significant structural damage.

A pseudo-static method has to be used for assessing the safety of the structure, the seismic action is represented by a set of horizontal and vertical static forces equal to the product of the gravity forces and a seismic coefficient.

The peak ground design acceleration has to be assumed according to the maximum value provided by the Geological Report, considering a reference period for the ground horizontal peak acceleration equal to 475 years and a proper soil factor.

The horizontal (k_H) and vertical (k_V) seismic coefficients affecting all the masses have to be evaluated as:

$k_H = \alpha \ S/r$

$k_V = +/- 0.5 \ k_H \ (a_{vg}/a_g) > 0.6$

where:

- α, is the ratio of the design ground acceleration, a_g, to the acceleration of gravity, g
- S, is the soil parameter (S=1)
- r, factor depending on the type of retaining structure (r=1 for rigid structure)

The components of the seismic forces have been calculated through seismic coefficients, k_H, k_V and applied to the relevant structural element according to the pseudo-static approach

- $F_H = k_h (\gamma_{ds} V)$, Horizontal Inertia forces of concrete elements
- $Fv = kv (\gamma_{ds} V)$, Vertical Inertia forces of concrete elements

  where V is the volume of the concrete element
- F_V,$= k_H (\gamma_{cushion} V)$ Horizontal Cushion layer Inertia force on the top slab
- $F_H = k_v (\gamma_{cushion} V)$ Vertical Cushion layer Inertia force on the top slab
- $\Delta SE = k_H \gamma H^2$ Backfill soil increment of pressure, according to Wood assuming a perfectly rigid wall

5.4.5 Retaining Wall

5.4.5.1 Dead load

Characteristic value of the cast in situ concrete is: 25 kN/m^3

5.4.5.2 Traffic load

Traffic load on the existing road has to be assumed equal to: 20 kN/m^3

5.4.5.3 Earth thrust

Earth pressure at rest is calculated based on the filling ground parameters, by means of the Jaky's formula:

K_0 = 1- sin (Φd)

where Φd = design angle of friction.

Active earth pressure is calculated based on the filling ground parameters, by means of the Rankine's theory.

5.4.5.4 Seismic load

See 5.4.3.5.

5.4.6 Excavation stabilisation

5.4.6.1 Self-weight

Characteristic value of the shotcrete self-weight is: 25 kN/m^3

The weight of reinforcement by wiremeshes is considered included.

5.4.6.2 Soil and rock thrust

The rock thrust acting on the excavation surface is the main load component for the excavation walls stabilization measures.

Weight, resistance and stiffness parameters shall be considered according to geotechnical-geomechanical evaluations (see chapter 4).

5.4.6.3 Earthquake load

Earthquake horizontal and vertical acceleration has to be considered.

Pseudo static analyses, with the peak horizontal acceleration expected for a given return period, will be performed to determine seismic stability of the excavation.

The peak value of ground horizontal acceleration for reference period of 475 years for the project area is a_g ***[Specify as per project area, ex. 0.31g].***

The response of ground slopes to the design earthquake can be calculated using pseudo-static methods of stability analysis, where the design seismic inertia forces Fh and Fv acting on the ground mass, are defined:

$$F_H = 0{,}5\,\alpha \cdot S \cdot W$$

$F_V = \pm 0{,}5\, F_H$ if the ratio a_{vg}/a_g is greater than

$F_V = \pm 0{,}33\, F_H$ if the ratio a_{vg}/a_g is not greater

Assuming S (soil factor) equal to unit, the value of peak ground acceleration is equal to 0.155g.

In the ratio between components of the seismic action is defined as a_V =67% a_H: this leads to a definition of design acceleration values, as:

- agH_permanent = **0.155 g**
- agV_permanent = **0.5 x agH = 0.078 g**

5.4.7 Culverts

5.4.7.1 Hydraulic dimensioning

<u>Precipitation data</u>

……………..”Insert Project Specific Details”……………………..………………………..

Example:

“Precipitation data of DHM’s Hydrological Station no. 703 at Butwal was adopted, according to value reported in ch. 3.3. Since the precipitation station is nearby the study area, it has been assumed that the characteristic precipitation of the area under study are correctly represented by the station recording.

Precipitation data of the selected station over 61 years, from 1957 to 2017 has been used in calculation. The data observed in this station is 24 hours accumulated rainfall over the year for 61 years; from the available data 24 hour maximum rainfall has been chosen for frequency analysis.”

<u>Design Intensity</u>

Considering the life period of the crossing structures, probable risk during heavy rainfall and overall investment on the construction, generally 25 year of return period intensity is adopted for crossing design, according to Nepal Road Safety Notes 2, “Design Safe Side Drains”.

The max forecasted intensity of 25 years of return period predicted by different common methods (Gumbel, log Person III, Log Normal) has been considered for analysis and design of crossing by-passes.

Exception to this assumption has been considered for watershed presenting very high possibility of debris flow, considering a 100 years flood, with about 40% extra space for debris flow (against the generally assumed 30% of extra space).

<u>Design Flood</u>

Design flood has been evaluated using rational method, commonly adopted for computing peak discharge for small basins.

To generally consider the debris presence, highlighted also by the site visit, and to guarantee an optimal working of crossing structures in the accidental case of partial obstruction, the design flood has been increased of 30%. Therefore, final design flood is:

$$Q_d = 1.30 \times Q_{catchment}$$

To ease construction stage at site, the cross-sections of culverts was uniformed as far as possible.

<u>Catchment Area</u>

Main catchment area have to be evaluated on the basis of the available topography survey map. Area intercludes between main catchment basins has to be evaluated on the basis of the topography study.

5.4.7.2 Structure dimensioning

<u>Dead Load</u>

Self-Weight

The self-weight of the structural members has been calculated assuming a characteristic unit weight of reinforced concrete of 25 kN/m^3.

Ground pressure on top slab

The equivalent distributed loads over the top slab have been determined assuming a design cushion "hfill" upon the top slab and backfill unit weight equal to γ = 20 kN/m^3 (dry unit weight). The resulting pressure is given by:

$P=\gamma H$

Surcharge on ground

The vertical pressure acting on ground level is evaluated considering the contribution of soil loading the ground level at the top slab level, assuming γ = 20 kN/m^3. For the assumed height the resulting pressure is given by:

$P=\gamma H$

Backfill horizontal earth pressure

Based on the vertical pressure due to the surcharge on ground, horizontal earth pressure acting on the wall has been calculated as hydrostatic pressure acting on the total wall height, according to equation:

$P = \gamma\ H(z)\ k_0$

Where :

γ = backfill unit weight (kN/m^3), assumed equal to = 20 kN/m^3

K0 = 1 – sin(Φ') = at rest coefficient of lateral earth pressure

Φ' = internal friction angle of the backfill

H = calculation total height of the wall

Internal water pressure

Vertical water pressure acting on the foundation slab has been considered according to the following equation:

$P=\gamma H$

assuming H equal to the whole internal free height of the culvert, to be on the safe side and G = 10 kN/m$^{3.}$

Horizontal earth pressure acting on the wall has been calculated as hydrostatic pressure acting on the total wall height, according to equation:

$P = \gamma\ H(z)$

<u>Live Load</u>

Traffic Load

According to Eurocode surface traffic load due to pedestrian has been considered as an uniform pressure acting on the top slab.

Snow Load

According to Nepal weather conditions, different climates according to altitude can be observe. Since according to temperature temperatures during the year except altitude 3000m,are generally greater than 0°C, snow load could been neglected.

Wind Load

Wind Load has not been considered.

Temperature

A temperature variation on the top slab, given by two components:

- A uniform temperature component ΔT_U = +/- 15°C
- A linearly varying temperature difference component along z-axis, ΔT_{MZ} = +/-5°

has been considered, assuming a thermal expansion coefficient for concrete equal to α= 1.2E^{-5} [1/°C]

Shrinkage

The shrinkage effects have been considered according to Standards approaches and a uniform temperature component ΔT, equal to: $\Delta T = \varepsilon_{sh} / \alpha$

has been applied to the concrete top slab, where the thermal expansion coefficient for concrete equal to α = 1.2 E-5 [1/°C], and &' is the shrinkage coefficient.

Seismic Load

According to EN 1998-1/5, the structure has to be designed to fulfil its function during and after an earthquake, without suffering significant structural damage.

A pseudo-static method has to be used for assessing the safety of the structure, the seismic action is represented by a set of horizontal and vertical static forces equal to the product of the gravity forces and a seismic coefficient.

The peak ground design acceleration has to be assumed according to the maximum value provided by the Geological Report, considering a reference period for the ground horizontal peak acceleration equal to 475 years and a proper soil factor.

The horizontal (k_H) and vertical (k_V) seismic coefficients affecting all the masses have to be evaluated as:

$k_H = \alpha \, S/r$

$k_V = +/- \, 0.5 \, k_H \, (a_{vg}/a_g) > 0.6$

where:

- α, is the ratio of the design ground acceleration, a_g, to the acceleration of gravity, g
- S, is the soil parameter (S=1, ground type A)
- r, factor depending on the type of retaining structure (r=1 for rigid structure)

5.5 Methods of Analyses

5.5.1 FEM Models

The structural design has to be performed by means of 2D and 3D finite elements model representing the arrangement of the structures to estimate the resulting internal actions and deformations of the different elements.

Generally, models has to be developed considering:

- mono-dimensional elements (beam element type)
- bi-dimensional elements (plate type)

Nodes between concrete elements are generally considered rigid connections.

Design loads have to be generally modelled as:

- point load on nodes
- uniform/variable forces on mono-dimensional elements;
- uniform pressure on bi-dimensional elements.

Soil-structure interaction has to be evaluated by means of elastic boundaries (linear or nonlinear), with properties defined according to geotechnical characterization.

Linear and non-linear analyses has to be performed.

5.6 Specific design criteria

5.6.1 Tunnel support measures

5.6.1.1.1 Unstable blocks scenario

Assuming the tunnel axis, rock discontinuities and joints orientations, the unstable blocks stability analyses and anchors verifications must be run.

It can be assumed that blocks with a volume < 1m^3 will detach during excavation operations or will be safely supported by the first shotcrete layer.

Joint dips and dip directions shall be evaluated, as well as joints properties.

The design of stabilization measures for tunnel excavation profile shall check the following conditions:

- Structural verification (STR) of all structural elements:
  - Internal resistance of reinforced shotcrete layer (shear, bending moment and punching);
  - Internal resistance of redistribution plates at the bolts head;
  - Internal resistance of bolts.
- Geotechnical verification (GEO) of all structural elements:
  - External resistance (anchorage) of bolts.

5.6.1.1.2 Loosening

Anchors verification

Considering the geometry of the primary lining, it shall be assumed that loosening will be supported by means of anchors, loaded by tensile forces.

It will be verified that:

- effective anchorage is beyond the loosened rock mass.
- the maximum applied design load is lower than both internal and external (friction) resistance, i.e.:
  - Structural verification (STR):

    Internal resistance of redistribution plates at the bolts head; Internal resistance of bolts.
  - Geotechnical verification (GEO): External resistance (anchorage) of bolts.

Shotcrete verification

It shall be assumed that shotcrete lining, reinforced with lattice girders, steel ribs, wire meshes or a combination of them, will support actions imposed by the rock mass. Punching, bending moments and shear load will be checked according to Eurocodes.

Tunnel face stability

Verification of tunnel face stability shall be provided according to the Kovari & Anagnostou theory.

Forepoling umbrella verification

Verification of the forepoling umbrella shall be provided, where foreseen.

Invert verification

Where necessary, invert can be adopted to face strong lateral pressures.

Shotcrete invert lining shall be verified as well, by means of bending moments and shear load verifications according to Eurocodes.

5.6.2 Tunnel final lining and technical buildings

Reinforced concrete lining bears the total loosening load. By means of a proper static model the following shall be verified according to Eurocodes:

- ULS (Ultimate limit states)
  - Bending moments (interaction domain MN);
  - Shear load.
- SLS (Serviceability limit states)
  - deformations;
  - cracking.

Asymmetry of the loosening load shall be considered in the analyses.

Structural and geotechnical verifications (STR/GEO) must be provided for the foundation of all structures.

5.6.3 Tunnel specific design basis

For the design basis regarding tunnel parts and elements such as inner space, safety systems, auxiliary systems and draining systems, please refer to the BASIC PREREQUISITES FOR TUNNEL OPERATION and MINIMUM SAFETY REQUIREMENT FOR ROAD TUNNEL section of the Employer's Requirements of the Contract Document.

5.6.4 Rockshed

By means of a proper static model the following shall be verified for the reinforced concrete elements according to Eurocodes:

- ULS (Ultimate limit states)
  - Bending moments (interaction domain MN);
  - Shear load.
- SLS (Serviceability limit states)
  - deformations;
  - cracking.

Structural and geotechnical verifications (STR/GEO) must be provided for the foundation of all structures. Moreover, the Global stability (EQU) of the slope have to be considered.

As for design approach, load combinations and partial safety factors, reference is made to previous point «design approach» (point 5.3). All verifications shall be in accordance to relevant Eurocodes.

5.6.4.1 Rockfall Energy

According to Rockfall analyses the design energy has been evaluated assuming that the calculation volume is equal to the characteristic volume ($V_{cal} = V_k$), if the rockfall trajectories do not show significant rebounds; otherwise it is equal to the average "factorised" volume ($V_{cal} = 1.5xV_m = 5m^3$ " Project Specific Input") if the rockfall trajectories show at least 2 significant rebounds.

Thus, it assumed that during the fall of the boulders from imported heights, where the boulder, due to the steep topography, bounces several times, these are shattered into elements equal in size to the average volumes of the boulders listed. This phenomenon is considered plausible considering the degree of interlocking of the rock mass and the conditions of the joints.

5.6.5 Retaining walls

Wall structure, micropiles and anchors shall be verified according to Eurocodes.

The design of permanent micropiles and anchors shall check the following conditions:

- Structural verification (STR):
  - Internal resistance of reinforced concrete (shear and bending moment);
  - Internal resistance of permanent anchors and micropiles.
- Geotechnical verification (GEO):
  - External resistance (anchorage) of permanent anchors and micropiles ;
  - Foundation verifications wherever necessary.
- Global stability (EQU) of the slope.

As for design approach, load combinations and partial safety factors, reference is made to previous point «design approach» (point 5.3). All verifications shall be in accordance to relevant Eurocodes.

5.6.6 Excavation walls stabilisation

The design of stabilization measures for permanent excavation walls along the road shall check the following conditions:

- Structural verification (STR) of all structural elements:
  - Internal resistance of reinforced shotcrete layer (shear, bending moment and punching);
  - Internal resistance of redistribution plates at the bolts head;
  - Internal resistance of permanent bolts
- Geotechnical verification (GEO) of all structural elements:
  - External resistance (anchorage) of permanent bolts
- Global stability (EQU) of the slope at every excavation phase.

As for design approach, load combinations and partial safety factors, reference is made to previous point «design approach» (point 5.3). All verifications shall be in accordance to relevant Eurocodes.

5.6.7 Culverts

5.6.7.1 Hydraulic design

To get verification of culverts, Manning's equation was applied:

$$Q_{max} = K_s \cdot A \cdot R^{2/3} \cdot j^{1/2}$$

Where:

- Q_{max} = maximum discharge through crossing structure (m^3/s);
- Ks = 1/n = Stickler coefficient;
- n = Manning coefficient;
- A = wet area (m^2);
- R = hydraulic radius (m) = A/P;
- P = wet perimeter (m);
- j = culvert longitudinal slope (m/m).

In table below, all adopted design parameters are resumed.

Table 24: Design and verification parameters

Parameter	Value
Return period (TR)	25 years
Rainfall time	24 hours
Rainfall distribution	LogNormal
Rainfall maximum height	Data analyses
Run-off coefficient (C)	0.80
Culvert percentage storage	80%
Manning coefficient (n)	From standard charts

5.6.7.2 Structure design

For structural verifications, by means of a proper static model the following shall be verified for the reinforced concrete elements according to Eurocodes:

- ULS (Ultimate limit states)
  - Bending moments (interaction domain MN);
  - Shear load.
- SLS (Serviceability limit states)
  - deformations;
  - cracking.

Structural and geotechnical verifications (STR/GEO) must be provided for the foundation of all structures. Moreover, the Global stability (EQU) of the slope have to be considered.

5.6.7.3 Additional protection measures

The construction of a protection wall between the road and the water pit to protect the road from boulder has to be envisaged together with proper measures to limit and, possibly, avoid the debris flow obstruction of these structures.

Proper maintenance has to be addressed to keep correctly working the structure.

To avoid erosion phenomena along the valley slope, proper protection measures from the localized water flow coming from the culverts outlet, have to be foreseen along the valley and mountain side.

6 ROAD ALIGNMENT DESIGN

……………………..”Insert Project Specific Information”…………..……………………………..

Example :

“The project, focused on the rockfall protection of the Siddhababa Section along the Siddhartha Highway, foresees both the rehabilitation of the existing road, from chainage 28+200.00 to 29+100.00 (called *Main Road 1 (MR1)*) and from chainage 30+050.00 to 30+600.00 (called *Main Road 2 (MR2), North)*, and the variation of the alignment lay-out to connect the new tunnel road approximately 1126 m long, by deviating from the old alignment towards East around chainage 29+100, near the existing «Siddhababa Mandir» and towards West around chainage 30+050, near the existing «Ramapithecus Park» .

The rehabilitation of the existing road has been developed and design to limit rock excavation along the mountain side and rock stabilization/earth retaining structures along the valley side, to reduce costs and time of construction.

An existing track diversion in a new two lanes tunnel for cars only (one lane for each direction), along the whole high hazard sections, departing from the existing road near the Siddha Baba Mandir and reconnecting to it near the hydropower plant, with a total length of about 1126 m, has been foreseen.

The new tunnel extends between chainages 0+908.46 (south portal) and 2+034.52 (north portal) and the position of the tunnel portals has been chosen considering a suitable site to minimize necessary excavation, hence entering the mountain almost perpendicularly to the slope, as well as to avoid the whole high hazard stretch.

The U-shape of the tunnel horizontal alignment has been chosen in order to reach as soon as possible a sufficient depth (tunnel overburden) to minimize the tunnel excavation in low overburden areas, where weaker and altered rock can be foreseen, as well as larger water income.

Bicycles and pedestrian keep on following the existing road alignment (called *Pedestrian Road (PE)*), but protected, along the identified high hazard section, by a rockshed with reduced internal dimensions of **5.0mx3.5m**, suitable to house only pedestrians and ambulance (or emergency car), in case of emergency situations. Thus, the existing road has been converted into a pedestrian-only road around chainage 29+100 and 30+050, comprehensive of two emergency aprons for reversal of traffic direction in case of tunnel closure during emergency. Moreover, each apron could be used as heliport in case of emergency. On the pedestrian road, the construction of a covered path with a rockshed has been foreseen from chainage around 29+215 to 29+995. The rockshed is connected to the three pedestrian by-passes foreseen along the tunnel, with a spacing smaller than 300m, so that, in case of emergency, the tunnel users can evacuate in the shortest possible time from the danger zone, leading directly to a safe space within the bypass tunnel itself and, from there, to the pedestrian rockshed along the existing road. No emergency escape route for vehicular access is foreseen due to the length of the tunnel (< 1.2 km).

The design parameters adopted for the road design of the Siddhababa stretch follow DoR Nepal Rural Road Standard (2070), 2013. Cross sections every 10m chainage have been considered to study and obtain the presented design alignment; however sections every 20m are reported.

The design of this pedestrian road and escape route does not undergo car traffic road design presented in the following paragraphs for the main road.

Please note that the general absolute chainage referred to the existing Siddhartha Highways has to be abandoned for this project due to the new alignment of the Siddhartha stretch, including the new tunnel road.

In the following are briefly summarized the road design main characteristics; for further details refer to the relevant specific reports.

Table 25:Principal stretches of main road

AXIS	TYPE OF INTERVENTION	ABSOLUTE CHAINAGE*		DESIGN CHAINAGE	
		From (km)	To (km)	From (km)	To (km)
Main road 1 (MR1)	Adjustment	28+200.00	29+104.55	0+000.00	0+904.55
Tunnel (MT)	Variation	29+104.55	30+050.00	0+904.55	2+034.52
Main road 2 (MR2)	Adjustment	30+050.00	30+600.00	2+034.52	2+587.40

* *Absolute chainages referred to the existing Siddhartha Highways must be abandoned; here are reported just as general reference, not to be adopted for design.*

Table 26:Principal stretches of pedestrian road

Pedestrian Road (PR)		
ZONE	ABSOLUTE CHAINAGE*	DESIGN CHAINAGE

	From (km)	From (km)	From km	To km
South apron	About 29+114	About 29+214	0+000.00	0+110.00
Rockshed	About 29+214	About 29+994	0+110.00	0+890.00
North apron	About 29+994	About 30+051	0+890.00	0+944.70

** Absolute chainages referred to the existing Siddhartha Highways must be abandoned; here are reported just as general reference, not to be adopted for design. "*

6.1 Road Classification

Road main features limits according to DoR Nepal Road Standard 2070 have to be assumed.

……………………."Insert Project Specific Information"………….……………………………..

Example :

"The design road falls under the category of National Highway[1] since the Siddhababa sections belongs to the Siddharta highway, which is a major highway in Nepal connecting the Terai region in southern Nepal with the mountain region in northern Nepal. The highway starts at Nepal–India border near Siddharthanagar and terminates at Pokhara. This highway intersects with the east–west Mahendra Highway at Butwal.

According to the technical and functional classification provided by Nepal Road Standard 2070, the design road falls within Class II [2], in mountainous and steep terrain, strongly depending on the harsh conditions of the surroundings and the alignment of the existing road."

6.2 Design Speed

The design speed has a crucial role in geometric parameters of the roads. The design speed depends on various factors like; super elevation, sight distance, radius and length of horizontal curve, extra widening of pavement, and the length of vertical curve (summit and valley) etc.

……………………."Insert Project Specific Information"………….……………………………..

Example :

"According to the design standards (Nepal Road Standard 2070), the ruling design speed adopted along the Siddhababa section of the rehabilitated road and the new road tunnel has been defined according is 40 km/h along the rehabilitated existing road stretches before and after the new tunnel, with exception of the road sections between chainages 2+320.00 and 2+420.00, where the design speed has to be limited to 25 km/h, according to the existing road geometrical alignment limits.

Indeed, within the tunnel, a design speed of 60 km/h has been adopted since this new road section can be designed according to the design speed for class II in mountainous terrain being a alignment; the upgraded speed of 60m/h at the south portal starts within the rock tunnel, at the begin of the tunnel road line."

6.3 Alignment Design Requirements

Main design parameters are briefly summarized in the following table, according to Nepal Road Standard 2070.

……………………..”Insert Project Specific Information”…………..……………………………..

Example:

"Table 27 Design parameters for Siddhababa section according to Nepal Road Standard 2070

S.N.	Design parameters	National Highway, Mountainous
1	Design Capacity - in both directions (Vehicle per day/P.C.U. per day)[3]	20000
2	Design speed (km/h)	40 (MR1 and MR2) 25 (MR2 from ch. 2+320.00 to 2+420) 60 (Tunnel)
3	Lanes width (m)	3.75
4	Carriageway width (m)	7.50
5	Shoulder width, either side (m)	0.50
6	Stopping distance (m)	50.00
7	Minimum radius of horizontal curves (m)	40.00
8	Maximum super elevation (%)	7.00
9	Minimum length of transition curves (m)	40.00

		9.00 (for 40 km/h) 7.00 (for 60 km/h)
10	Maximum gradient (%)	200.00 (for 9.00%)
11	Maximum length of gradient(m)	300.00 (for 7.00%) 29.00 (for 40 km/h)
12	Minimum value of K for summit curves (m/%)	94.00 (for 60 km/h)
13	Minimum value of K for valley curves (m/%)	17.00 (for 40 km/h); 42.00 (for 60 km/h)

"

6.3.1 Excavation and fill

Excavation and Fill Excavation has been balanced and limited as far as possible.

6.4 Safety Barriers, Pavement Marking and Traffic Signs

Dimensioning of safety barrier shall be provided according to the requirements of Nepal Safety Barrier, 1997. Marking/Signing shall be provided accordingly to Nepal Road Standard (2070), Nepal Traffic Sign Manual, 1997

6.4.1 Additional requirements for Tunnel Road alignment

For additional requirements for the Tunnel Road alignment please refer to the BASIC PREREQUISITES FOR TUNNEL OPERATION and MINIMUM SAFETY REQUIREMENT FOR ROAD TUNNEL section, and other relevant sections of the Employer's Requirements of the Contract Document.

6.5 Pavement Design

6.5.1 Design Life

According to Nepal Design Guidelines for flexible pavement 15 years have to be considered for highways pavement. In addition 5 years have been considered to account for the estimated design and construction time, so that a total design life of 20 years.

6.5.2 Traffic Study

According to traffic data provided by the Client and presented in ch. 3.1, all traffic in the classified count has been classified into the following types of vehicles as per the DoR practice:

- Heavy truck (three axles or more);
- Heavy two axles;
- Mini truck/tractor;
- Large bus;
- Bus.

For analytical purpose, the AADT at the road is expressed both in terms of vehicle per day (CVPD) and daily passenger unit (PCU/day):

……………………..”Insert Project Specific Information”…………..……………………………..

Example:

Table 28:Traffic data according to survey on site

Traffic data		
AADT	12444	CVPD

AADT excluding motorcycle and rickshaw	3336	CVPD
AADT	9478	PCU
AADT in PCU excluding motorcycle and rickshaw	4912	PCU

The traffic spectrum derived from survey is:

Table 29: Traffic spectrum according to survey on site

Vehicle type	%
Heavy truck (three axles or more)	0.93%
Heavy two axles	8.28%
Mini-truck/tractor	0.85%
Large bus	9.80%
Bus	8.97%

“

According to Nepal Road Standard 2070, the following converting factors has been assumed to correlate PCU to AADT,

Table 4-1 Vehicle types, Equivalency Factors

SN	Vehicle Type	Equivale
4	Bicycle,Motorcycle	(
1	Car, Auto Rickshaw, SUV,Light Van and Pick Up	1
2	Light (Mini) Truck, Tractor, Rickshaw	1
3	Truck,Bus,Minibus,Tractor with trailer	3

The traffic forecast is computed according the growth formula mentioned in the IRC:37-2018 :Guidelines for The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS

6.5.2.2 Vehicle Damage Factor (Vdf)

For Vdf, IRC:37-2018 :Guidelines for The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS relevant clause shall be followed.

6.5.2.3 Lane Distribution Factor (D)

Total traffic AADT (both way) is distributed over the whole carriageway for design of pavement. For D, IRC:37-2018 :Guidelines for The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS relevant clause shall be followed.

6.5.2.4 Design Traffic

The design traffic load in terms of cumulative number of standard axles (ESA) shall be calculated as per IRC:37-2018 :Guidelines for The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS relevant clause.

6.5.3 Subgrade properties

……………………..”Insert Project Specific Information”…………..……………………………..

Example :

The proposed pavement for the main road has been designed considering a subgrade made of the rock outcrop according to properties detailed in the geotechnical report.

Where the rock is not present as a sub-grade and it is necessary to fill it with granular material, it must have a minimum CBR = 5 (E = 50 MPa) as the bearing capacity.

6.5.4 Pavement Design

The pavement design for both roadway section and tunnel section shall be designed as per IRC:37-2018 :Guidelines for The Design of Flexible Pavements, (Fourth Revision),INDIAN ROADS CONGRESS relevant clause.

6.6 Road water management system

6.6.1 Rainfall data

Rainfall data and catchment area are assumed according to data reported in ch. 3.3

6.6.2 Main Road

For surface water management, the mountain side camber principle has to be adopted for proper management of surface water and side drains on the road mountain side has been adopted along the whole road stretches.

Side drains are assumed to receive the road surface water and the water coming from next mountain ridge that is not regimented by culverts.

A carriageable concrete cover has to be foreseen on side drains, allowing them to be used as walkaways in order to reduce the road total width, thus limiting earth works and rock excavation.

6.6.3 Tunnel Road

Dirty and polluted waters coming from the inner tunnel space are collected by a linear drainage channel, thanks to the transversal gradient of the carriageway and transported to the first chamber of the syphoned wells.

Special provisions has to be considered to allow to avoid direct communication between the inner space of the tunnel and the main water collector and, hence, avoid propagation of fire liquids along the tunnel in case of accidents/fire.

Since some waters could also leak under the carriageway and seep through the filling gravel, a dedicated drainage system has to be considered

Water basin, that allows to collect waters and, whenever necessary, to avoid the direct discharge into the local sewage systems has to be considered

6.6.4 Pedestrian Road

Before and after pedestrian rockshed, proper side drains has to be foreseen, considering a carriageable cover also for pedestrian traffic, where they have to be used as walkaways such as at the tunnel south portal near the Siddhaba Temple.

Along the road sections interested by the rockshed structure, a proper drainage system has to be designed for the possible rain water incoming due to the wall openings along the valley side.

A proper drainage system of the cushion layer and backfill materials due to water coming from the mountain, not collected into culverts has to be foreseen.

7 ROCKFALL MITIGATION MEASURES DESIGN

7.1 Materials

7.1.1 Rockfall protection Barriers (Geobrug RXE type or equivalent)

<u>a. Barrier type GEOBRUGG RXE 2000 or similar</u>

- Absorption capacity 2000 kJ
- Energy class EOTA 5
- Height 4 m
- Post spacing 8-12 m (average 10 m)

<u>b. Barrier type GEOBRUGG RXE 3000 or similar</u>

- Absorption capacity 3000 kJ
- Energy class EOTA 6
- Height 4 m
- Post spacing 8-12 m (average 10 m)

<u>c. Barrier type GEOBRUGG RXE 5000 or similar</u>

- Absorption capacity 5000 kJ
- Energy class EOTA 8
- Height 5 m
- Post spacing 8-12 m (average 10 m)

7.1.2 Adherence Net system (Tecco type)

<u>Adherence net</u>

High-tensile steel wiremesh type Tecco G65/4 or equivalent

Rock bolts

- Type: Gewi diam. 32 mm (steel grade B500B) or equivalent
- Tensile Strength Ftk ≥ 466 kN
- Yield Strength Fyk ≥ 402 kN
- Drilling diameter ≥ 110 mm

7.1.3 Additional measures for mudstone layers

Permanent shotcrete:

- Reference Code: UNI EN 1992 1-1, EN 206-1
- Concrete Class: C30/37
- Characteristic Cubic Strength: Rck >37 MPa
- W/C: 0.5
- Cement Type: III-IV (UNI EN 197-1)
- Exposure Class: XC4/XD1/XF1
- Resistant at AAR
- Slump Class: S4
- Maximum Aggregate: 8 mm (EN 12620)

Steel wiremesh (shotcrete reinforcement):

- Bar diameter, D = 6mm
- Mesh spacing = (100x100)mm
- Steel grade min. B500 A
- Yield Strength fyk = 500 MPa

Rockbolts:

- Type: Permanent Gewi diam. 28 mm (steel grade B500B) with preinjected corrugated sheating or equivalent
- Tensile Strength Ftk ≥ 357 kN
- Yield Strength Fyk ≥ 308 kN
- Drilling diameter ≥ 110 mm

7.2 Design Working Life

See previous para. 5.2

7.3 Design Approach

See previous para. 5.3

7.4 Design Loads

7.4.1 Rockfall protection barriers

Rockfall protection barriers shall be designed in order to withstand the design impact energy foreseen by rockfall simulations, based also on the design rock block dimensions.

All structural elements, including foundation micropiles and retaining anchors, shall be dimensioned to withstand actions consequent to such impact events.

Reference to the rockfall barrier supplier technical specifications shall be also made.

7.4.2 Slope stabilization systems (adherence mesh and shotcrete layer)

The adherence net as well as the shotcrete layer on mudstone layers must be designed to stabilize superficial blocks that can be unstable between provided bolts, accordingly to performed geomechanical characterization of the slope.

As for the rock bolts, they must be dimensioned considering different load conditions:

- to support, by tension and shear, the adherence nets and shotcrete layers, assuming the above mentioned load condition;
- to support, by shear, possible unstable blocks, accordingly to performed geomechanical characterization of the slope

7.4.3 Slope below road works

Rock stabilization shall be provided where highly fractured rock is observed on the slope below the existing road, where erosion and weathering cause the gradual fall of boulders and/or detachment of rock slabs.

Anchors shall be designed to stabilize superficial blocks or slabs that can be unstable.

They are dimensioned considering to support, by shear, possible unstable blocks/slabs.

As unstable block, the average block dimension can be considered, as resulting by geological mapping and survey, and the main joint families orientation shall be considered in order to estimate block shape.

As example, based on such analysis, the design block estimated in the project has a volume of 10 m^3/m. Such assumption shall be checked and confirmed or revised by Tenderer/Contractor based on its own evaluation and possible additional surveys.

Shotcrete layer design follows what suggested in 7.4.2

7.5 Specific design criteria

7.5.1 Rockfall protection barriers

The design of slope stabilization systems shall check the following types of failure/resistances:

- **Protection barrier**
  - Structural verification according to the supplier certified system, based on impact energy.
- **Anchors and micropiles (rope anchorage and barrier foundation):**
  - Structural verification (STR) of all structural elements:
    - Internal resistance of anchors and micropiles (axial and shear).

- O Geotechnical verification (GEO) of all structural elements:
  - External resistance of anchors and micropiles.
- **Global stability (EQU) of the existing slope considering the load transfer by the barrier foundation micropiles to the rock.**

7.5.2 Slope stabilization systems (adherence mesh and shotcrete layer)

The design of slope stabilization systems shall check the following types of failure/resistances:

- **adherence mesh system (Tecco net type of equivalent and its bolts) :**
  - O Structural verification (STR) of all structural elements:
    - Internal resistance of the net itself, including redistribution plates;
    - Internal resistance of the perimetric rope ;
    - Internal resistance of perimetric wire anchors;
    - Internal resistance of bolts (axial and shear).
  - O Geotechnical verification (GEO) of all structural elements:
    - External resistance (anchorage) of perimetric wire anchors
    - External resistance (anchorage) of bolts ;
- **Shotcrete layer on mudstone layers and its bolts :**
  - O Structural verification (STR) of all structural elements:
    - Internal resistance of the reinforced shotcrete (shear, bending moment and punching);
    - Internal resistance of redistribution plates at the bolts head;
    - Internal resistance of bolts (axial and shear).
  - O Geotechnical verification (GEO) of all structural elements:
    - External resistance (anchorage) of bolts ;
- **Global stability (EQU) of the existing slope.**

As for design approach, load combinations and partial safety factors, reference is made to previous point «design approach» (point 5.3); all verifications shall be in accordance to relevant Eurocodes.

7.5.3 Slope below road works

The design of slope stabilization systems shall check the following types of failure/resistances:

- **Shotcrete and Anchors:**
  - O Structural verification (STR) of all structural elements:
    - Internal resistance of the reinforced shotcrete (shear, bending moment and punching);
    - Internal resistance of redistribution plates at the bolts head;
    - Internal resistance of bolts (axial and shear).

- Geotechnical verification (GEO) of all structural elements:
  - External resistance (anchorage) of bolts;
- **Global stability (EQU) of the existing slope (existing situation and final situation with rockshed loads).**

As for design approach, load combinations and partial safety factors, reference is made to previous point «design approach» (point 5.3); all verifications shall be in accordance to relevant Eurocodes.