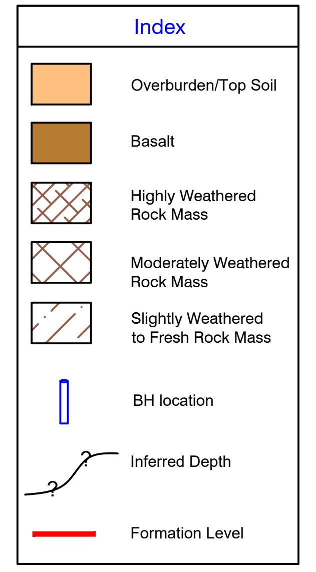
ANNEXURE- IV

Field Photographs

ANNEXURE- V

Geological L- Sections



PROPOSED F.L	553.888	553.803	553.717	553.632	553.547	553.462	553.377	553.292	553.207	553.122	553.037	552.951
GROUND LEVEL	563.188	565.910	572.502	573.175	573.131	573.570	574.376	574.338	574.088	574.046	573.320	573.567
CHAINAGE	7500	7520	7540	7560	7580	7600	7620	7640	7660	7680	7700	7720

PROPOSED F.L	552.866	552.781	552.696	552.611	552.526	552.441	552.356	552.271	552.185	552.100	552.015	551.930
GROUND LEVEL	574.347	575.117	577.066	586.181	585.607	586.719	581.136	586.117	582.064	574.206	571.017	571.133
CHAINAGE	7740	7760	7780	7800	7820	7840	7860	7880	7900	7920	7940	7960

PROPOSED F.L	551.845	551.760	551.675	551.590	551.505	551.420	551.334	551.249	551.164	551.079	550.994	550.909
GROUND LEVEL	573.861	582.051	587.033	587.005	578.969	581.160	580.539	583.304	585.559	586.414	587.014	587.122
CHAINAGE	7980	8000	8020	8040	8060	8080	8100	8120	8140	8160	8180	8200

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Project
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Tunnel No. 1

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Date NOV. 2024	Sheet 1of 6	Revision 0
Drawing No: RITES/WR/RK/GT/T1		

CHAINAGE	8200	8220	8240	8260	8280	8300	8320	8340	8360	8380	8400	8420
PROPOSED F.L	550.909	550.824	550.739	550.654	550.568	550.483	550.398	550.313	550.228	550.143	550.058	549.973
GROUND LEVEL	587.122	587.788	586.977	587.564	587.493	587.163	590.610	595.609	591.585	588.947	597.858	599.014

CHAINAGE	8440	8460	8480	8500	8520	8540	8560	8580	8600	8620	8640	8660
PROPOSED F.L	549.888	549.802	549.717	549.632	549.547	549.462	549.377	549.292	549.207	549.122	549.036	548.951
GROUND LEVEL	599.750	599.914	599.631	599.571	600.583	601.439	602.144	602.847	602.771	603.570	603.590	603.533

CHAINAGE	8680	8700	8720	8740	8760	8780	8800	8820	8840	8860	8880	8900
PROPOSED F.L	548.866	548.781	548.696	548.611	548.526	548.441	548.356	548.271	548.185	548.100	548.015	547.930
GROUND LEVEL	603.311	602.545	601.623	600.621	600.491	600.184	599.733	600.941	601.447	599.567	597.926	596.619

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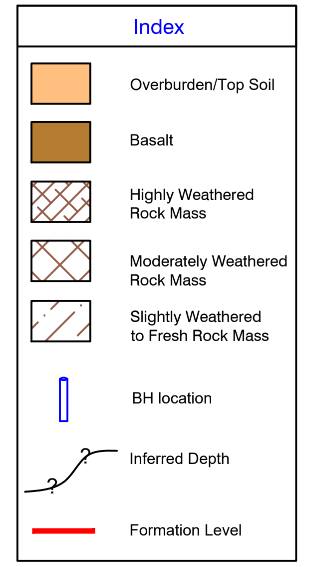
Date	NOV. 2024	Sheet	2of 6	Revision
Drawing No: RITES/WR/RK/GT/T1				0

PROPOSED F.L	547.930	547.845	547.760	547.675	547.590	547.505	547.419	547.334	547.249	547.164	547.079	546.994	546.909	546.824	546.739	546.653	546.568	546.483
GROUND LEVEL	596.619	596.502	598.145	600.178	601.339	602.579	604.204	602.022	599.266	598.864	599.858	601.465	603.792	604.451	606.152	604.967	601.146	599.310
CHAINAGE	8900	8920	8940	8960	8980	9000	9020	9040	9060	9080	9100	9120	9140	9160	9180	9200	9220	9240

PROPOSED F.L	546.398	546.313	546.228	546.143	546.058	545.973	545.888	545.802	545.717	545.632	545.547	545.462	545.377	545.292	545.207	545.122	545.036	544.951
GROUND LEVEL	596.275	597.235	600.472	599.339	599.681	603.763	610.972	612.799	610.626	609.179	606.223	604.600	605.271	609.763	612.064	612.525	610.569	615.113
CHAINAGE	9260	9280	9300	9320	9340	9360	9380	9400	9420	9440	9460	9480	9500	9520	9540	9560	9580	9600



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Final Location Survey and Geotechnical Investigation for BG line in connection with gauge conversion of existing Ratlam - Khandwa MG section of Western Railway

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Date	NOV. 2024	Sheet	3of 6	Revision 0
Drawing No: RITES/WR/RK/GT/T1				

CHAINAGE	9600	9620	9640	9660	9680	9700	9720	9740	9760	9780	9800	9820	9840	9860	9880	9900	9920	9940
PROPOSED F.L	544.951	544.866	544.781	544.696	544.611	544.526	544.441	544.356	544.270	544.185	544.100	544.015	543.930	543.845	543.760	543.675	543.590	543.504
GROUND LEVEL	615.113	615.137	615.352	616.467	617.757	618.143	616.209	614.312	615.901	616.968	614.659	614.932	609.776	610.707	605.631	603.980	601.582	605.797

CHAINAGE	9960	9980	10000	10020	10040	10060	10080	10100	10120	10140	10160	10180	10200	10220	10240	10260	10280	10300
PROPOSED F.L	543.419	543.334	543.249	543.164	543.079	542.994	542.909	542.824	542.739	542.653	542.568	542.483	542.398	542.313	542.228	542.143	542.058	541.973
GROUND LEVEL	612.969	616.271	617.278	614.966	611.206	613.751	614.522	614.571	614.577	616.796	615.323	610.074	610.125	610.428	613.989	617.586	617.458	618.186

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Date	NOV. 2024	Sheet	4of 6	Revision
Drawing No: RITES/WR/RK/GT/T1				0

PROPOSED F.L	541.973	541.887	541.802	541.717	541.632	541.547	541.462	541.377	541.292	541.207	541.121	541.036
GROUND LEVEL	618.186	619.082	619.998	620.181	620.218	620.604	620.838	620.922	621.071	621.268	621.652	621.993
CHAINAGE	10300	10320	10340	10360	10380	10400	10420	10440	10460	10480	10500	10520

PROPOSED F.L	540.951	540.866	540.781	540.696	540.611	540.526	540.441	540.355	540.270	540.185	540.100	540.015
GROUND LEVEL	622.390	622.870	622.848	622.840	623.166	623.469	623.626	623.957	623.963	623.766	624.014	624.024
CHAINAGE	10540	10560	10580	10600	10620	10640	10660	10680	10700	10720	10740	10760

PROPOSED F.L	539.930	539.845	539.760	539.675	539.590	539.504	539.419	539.334	539.249	539.164	539.079	538.994
GROUND LEVEL	624.208	624.158	624.272	624.923	625.931	628.704	633.377	634.848	634.110	633.300	633.200	633.806
CHAINAGE	10780	10800	10820	10840	10860	10880	10900	10920	10940	10960	10980	11000

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Geological L-Section Tunnel No. 1

Scale 0 10 20 30

Date	NOV. 2024	Sheet	5of 6	Revision
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PROPOSED F.L	538.994	538.909	538.824	538.738	538.653	538.568	538.483	538.398	538.313	538.228	538.143	538.058	537.972	537.887	537.802	537.717	537.632	537.547
GROUND LEVEL	633.806	634.410	634.562	634.890	635.201	635.405	635.483	635.790	636.086	636.548	637.003	637.686	638.087	638.829	640.278	642.648	641.306	638.510
CHAINAGE	11000	11020	11040	11060	11080	11100	11120	11140	11160	11180	11200	11220	11240	11260	11280	11300	11320	11340

PROPOSED F.L	537.462	537.377	537.292	537.206	537.121	537.036	536.951	536.866	536.781	536.696	536.611	536.526	536.441	536.355	536.270	536.185	536.100	536.015
GROUND LEVEL	628.365	622.121	618.520	610.747	601.860	593.572	586.372	579.952	574.010	570.882	569.764	569.650	564.005	556.275	554.240	553.586	550.377	546.357
CHAINAGE	11360	11380	11400	11420	11440	11460	11480	11500	11520	11540	11560	11580	11600	11620	11640	11660	11680	11700

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Title: Geological L-Section Tunnel No. 1

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Date NOV. 2024	Sheet 6of 6	Revision
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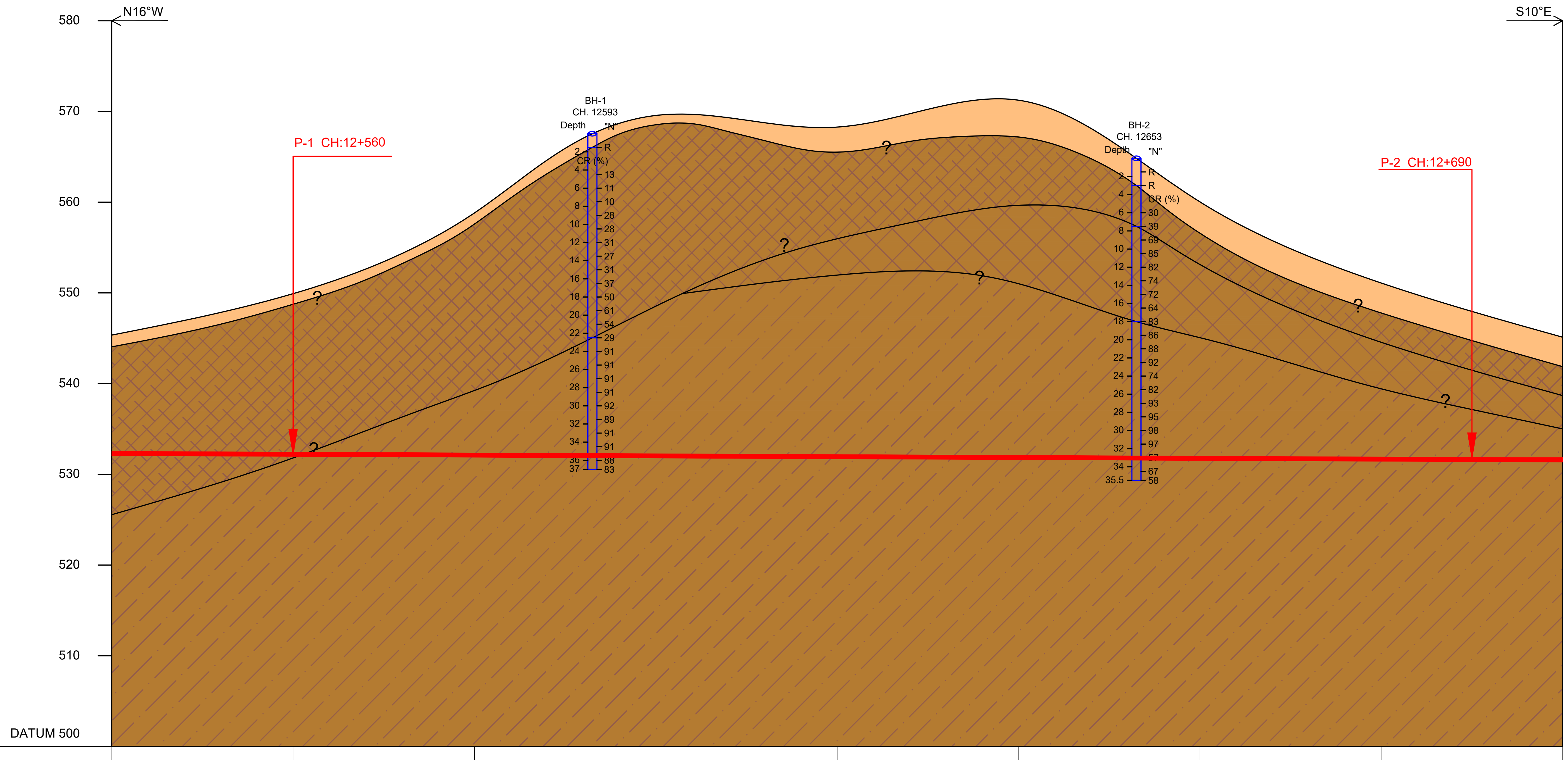

PROPOSED F.L	532.299	532.210	532.121	532.032	531.943	531.854	531.765	531.676	531.588
GROUND LEVEL	545.351	549.917	558.820	569.564	568.281	571.244	560.035	551.089	545.115
CHAINAGE	12540	12560	12580	12600	12620	12640	12660	12680	12700

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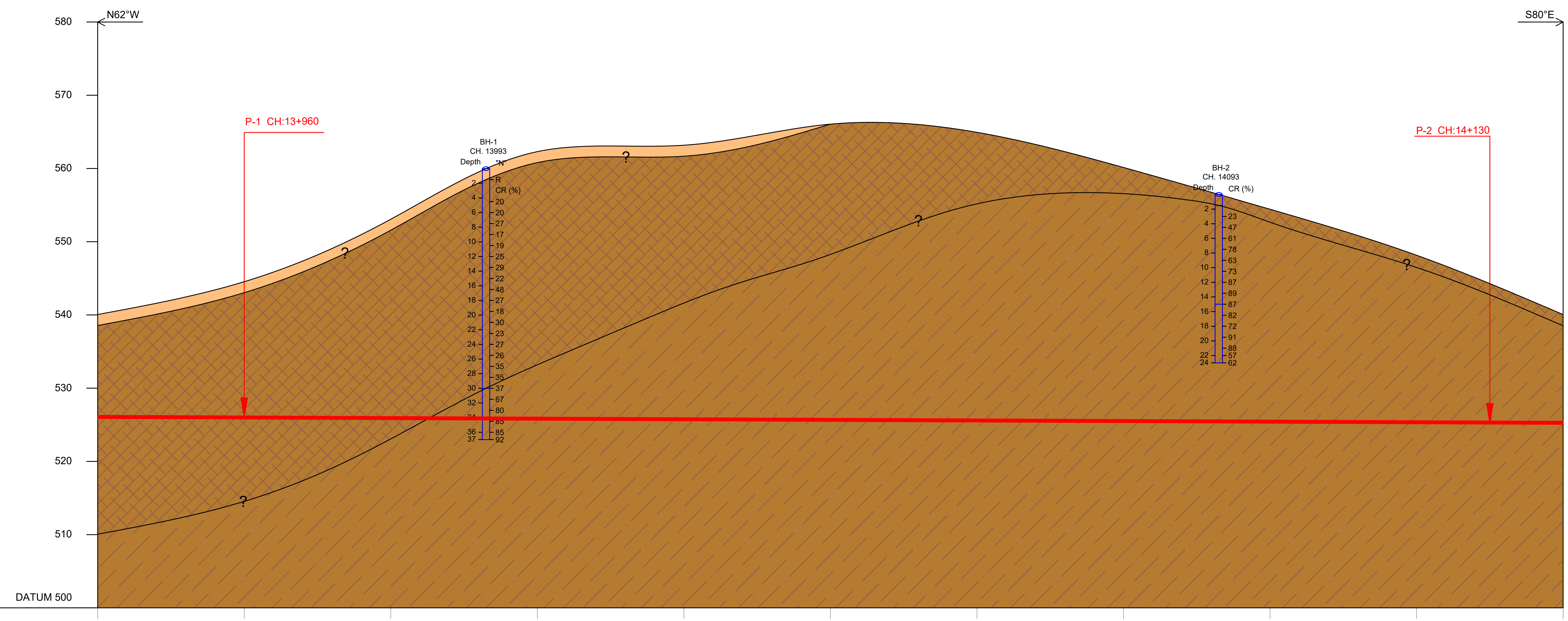
Geological L-Section
Tunnel No. 1A

Scale 0 10 20 30

Date NOV. 2024

Revision 0

Drawing No: RITES/WR/RK/GT/T1A



PROPOSED F.L	526.076	525.988	525.899	525.810	525.721	525.632	525.543	525.454	525.368	525.318	525.268
GROUND LEVEL	540.059	544.540	553.068	562.299	563.175	566.055	564.940	560.140	554.435	548.154	540.054
CHAINAGE	13940	13960	13980	14000	14020	14040	14060	14080	14100	14120	14140

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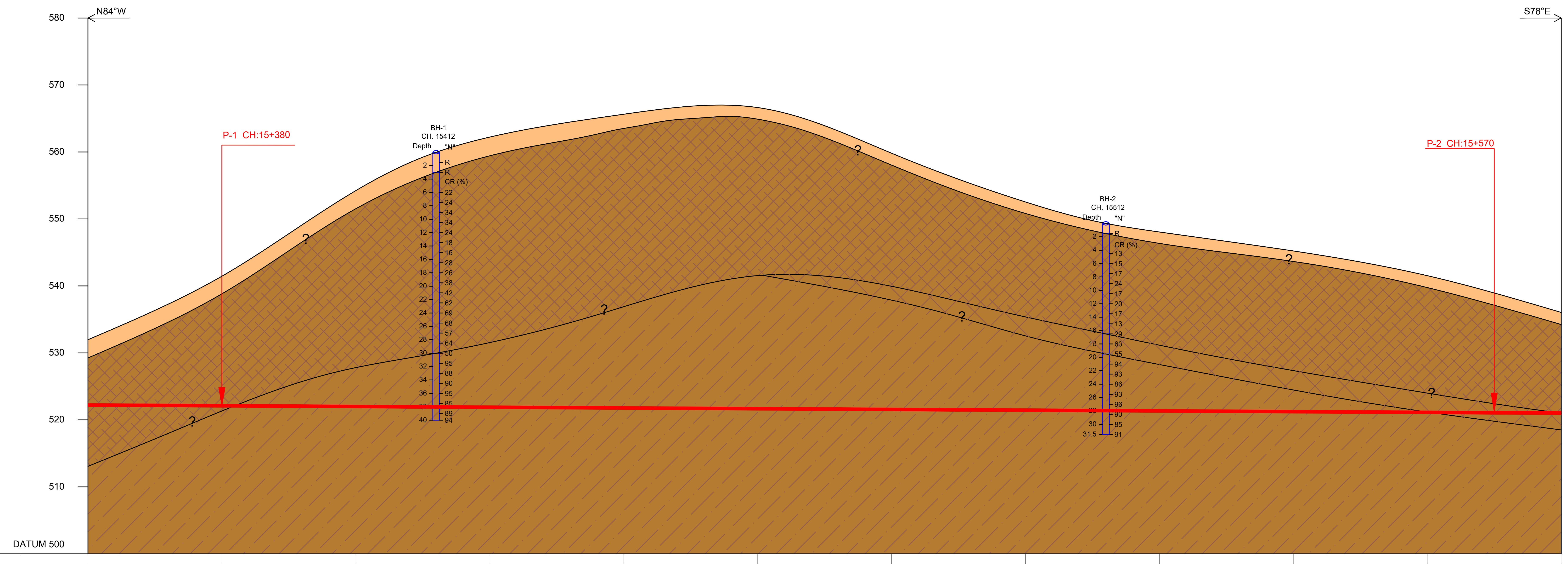
Geological L-Section
Tunnel No. 1B

Scale 0 10 20 30

Date NOV. 2024

Revision 0

Drawing No: RITES/WR/RK/GT/T1B



PROPOSED F.L	522.218	522.168	522.118	522.012	521.887	521.762	521.637	521.512	521.387	521.262	521.137	521.012
GROUND LEVEL	531.974	541.452	554.232	562.222	565.632	566.630	559.815	552.535	548.052	545.222	541.534	536.046
CHAINAGE	15360	15380	15400	15420	15440	15460	15480	15500	15520	15540	15560	15580

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Geological L-Section
Tunnel No. 2

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Revision 0

Drawing No: RITES/WR/RK/GT/T2

PROPOSED F.L	518.377	518.262	518.147	518.032	517.917	517.802	517.687	517.572	517.457	517.342	517.227
GROUND LEVEL	532.254	537.980	539.344	539.808	541.544	552.231	562.267	564.992	566.089	562.676	557.984
CHAINAGE	16000	16020	16040	16060	16080	16100	16120	16140	16160	16180	16200

PROPOSED F.L	517.112	516.997	516.882	516.768	516.653	516.538	516.423	516.308	516.193
GROUND LEVEL	561.789	556.003	557.531	558.224	558.894	550.965	542.761	535.003	529.149
CHAINAGE	16220	16240	16260	16280	16300	16320	16340	16360	16380

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Geological L-Section Tunnel No. 3

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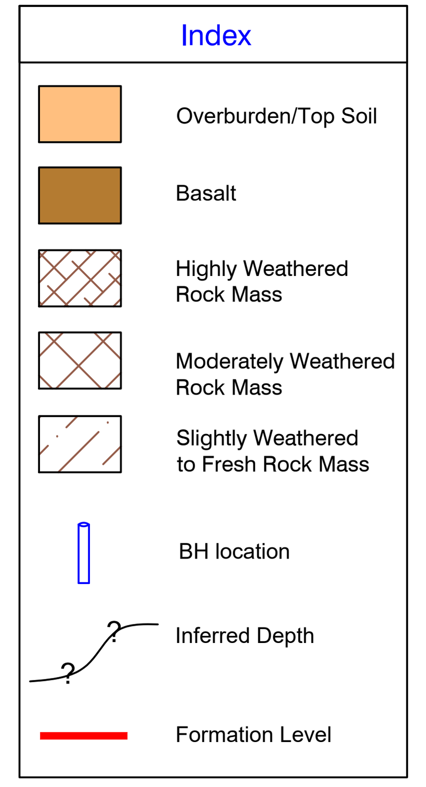
Drawing No: RITES/WR/RK/GT/T3

PROPOSED F.L	513.003	512.903	512.786	512.669	512.552	512.435	512.318	512.201	512.084	511.967	511.850	511.733	511.616	511.499	511.382
GROUND LEVEL	521.744	532.402	546.958	555.882	561.674	567.562	568.912	565.123	560.691	553.025	545.787	545.724	545.201	545.339	549.478
CHAINAGE	17000	17020	17040	17060	17080	17100	17120	17140	17160	17180	17200	17220	17240	17260	17280

PROPOSED F.L	511.265	511.148	511.031	510.914	510.797	510.680	510.572	510.472	510.372	510.272	510.172	510.072	509.972	509.872
GROUND LEVEL	552.141	558.602	566.550	574.342	573.025	579.644	582.497	574.510	568.733	561.927	554.953	546.030	535.200	522.219
CHAINAGE	17300	17320	17340	17360	17380	17400	17420	17440	17460	17480	17500	17520	17540	17560



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Geological L-Section Tunnel No. 4

Scale 0 10 20 30

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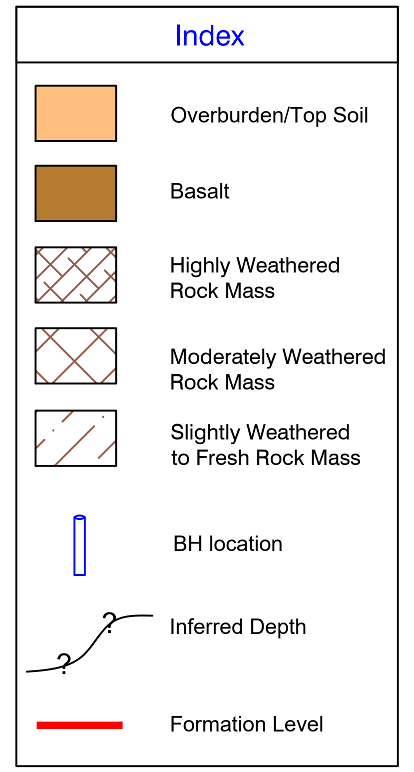
Drawing No: RITES/WR/RK/GT/T4

Revision 0

PROPOSED F.L	504.447	504.330	504.213	504.096	503.979	503.862	503.745	503.628	503.511	503.394	503.277	503.160
GROUND LEVEL	511.077	522.686	537.218	550.277	559.481	567.779	577.386	574.021	567.883	562.158	558.533	556.688
CHAINAGE	18640	18660	18680	18700	18720	18740	18760	18780	18800	18820	18840	18860

PROPOSED F.L	503.043	502.926	502.809	502.692	502.575	502.458	502.341	502.224	502.107	501.991	501.874	501.757
GROUND LEVEL	552.781	547.382	540.073	535.201	533.619	531.750	530.149	529.771	528.314	526.781	522.848	515.137
CHAINAGE	18880	18900	18920	18940	18960	18980	19000	19020	19040	19060	19080	19100



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Final Location Survey and Geotechnical Investigation for BG line in connection with gauge conversion of existing Ratlam - Khandwa MG section of Western Railway

Title

Geological L-Section
Tunnel No. 5

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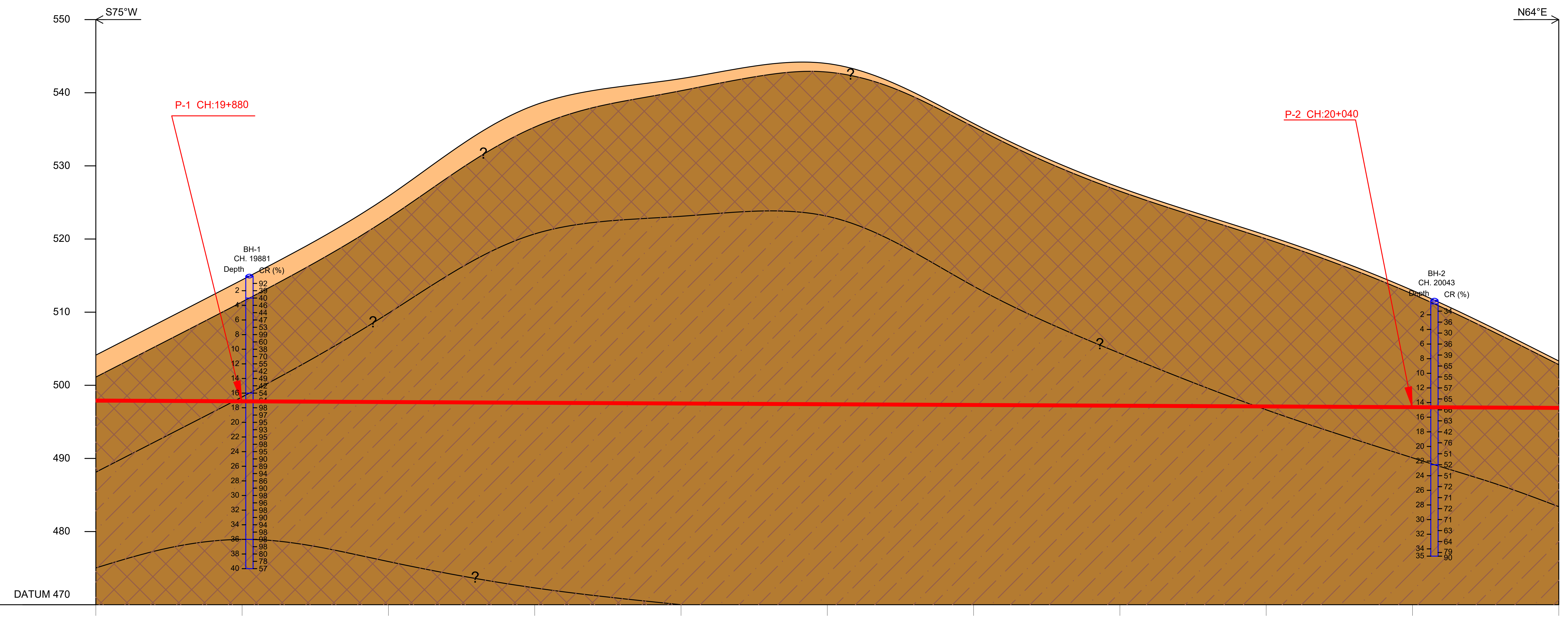
Drawing No: RITES/WR/RK/GT/T5



PROPOSED F.L	497.910	497.810	497.710	497.610	497.510	497.410	497.310	497.210	497.110	497.010	496.910
GROUND LEVEL	504.122	514.413	525.845	538.305	541.929	543.989	535.795	527.037	520.523	512.974	503.321
CHAINAGE	19860	19880	19900	19920	19940	19960	19980	20000	20020	20040	20060

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Title: Geological L-Section Tunnel No. 5A

Scale: 0 10 20 30

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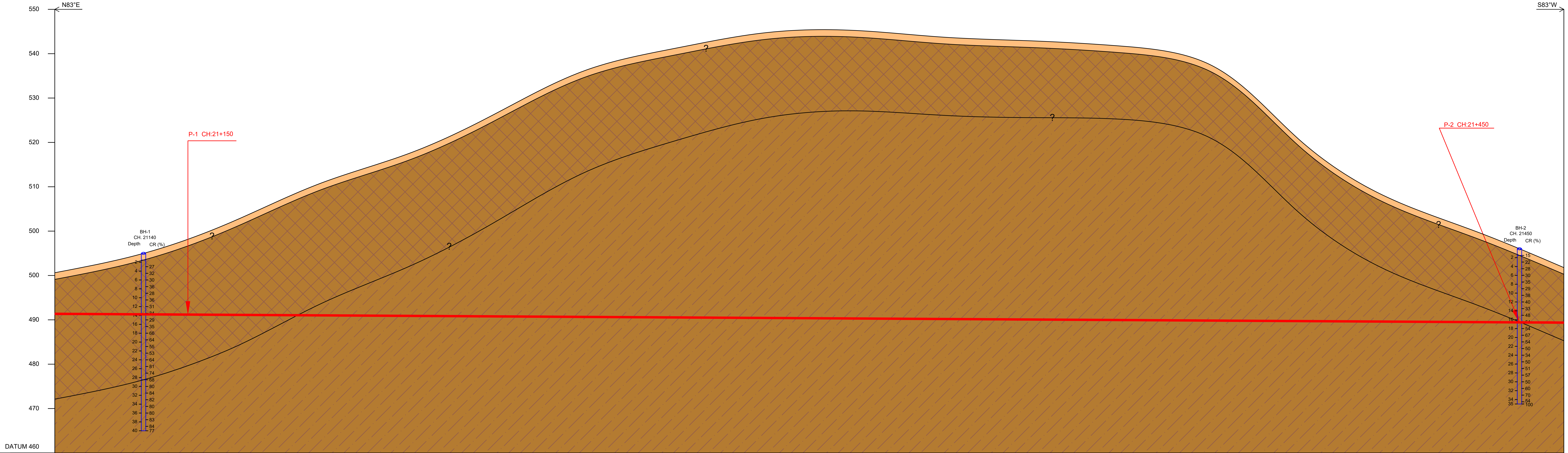
Revision: 0

Drawing No: RITES/WR/RK/GT/T5A



	21120	21140	21160	21180	21200	21220	21240	21260	21280
PROPOSED F.L	491.356	491.239	491.122	491.005	490.888	490.771	490.654	490.537	490.420
GROUND LEVEL	500.621	504.985	512.124	520.830	527.583	538.852	548.403	551.261	554.716
CHAINAGE	21120	21140	21160	21180	21200	21220	21240	21260	21280

	21300	21320	21340	21360	21380	21400	21420	21440	21460
PROPOSED F.L	490.303	490.186	490.069	489.952	489.835	489.718	489.601	489.484	489.367
GROUND LEVEL	555.259	553.743	552.899	551.740	547.674	530.963	517.739	509.997	501.781
CHAINAGE	21300	21320	21340	21360	21380	21400	21420	21440	21460

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Consultant: RITES

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Title: Geological L-Section Tunnel No. 6

Scale: 0 10 20 30

Date: NOV. 2024

Revision: 0

Drawing No: RITES/WR/RK/GT/T6

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PROPOSED F.L	483.321	483.187	483.054	482.921	482.787	482.654	482.521	482.387	482.254	482.121	481.987	481.854	481.721	481.587	481.454	481.320	481.187
GROUND LEVEL	492.275	501.008	510.763	522.730	535.777	545.293	549.596	551.876	550.434	548.812	548.389	541.711	530.356	520.340	509.498	501.014	494.089
CHAINAGE	22560	22580	22600	22620	22640	22660	22680	22700	22720	22740	22760	22780	22800	22820	22840	22860	22880

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Title

Geological L-Section Tunnel No. 7

Scale 0 10 20 30

Date NOV. 2024

Revision 0

Drawing No: RITES/WR/RK/GT/T7

PROPOSED F.L	460.928	460.867	460.805	460.744	460.682	460.62	460.559	460.497	460.436	460.374	460.313	460.251	460.19	460.128	460.067	460.005	459.944	459.882	459.82	459.759	459.697	459.636	459.574	459.513	459.451
GROUND LEVEL	478.399	488.521	496.094	505.826	518.995	536.277	545.231	553.795	558.762	548.901	539.105	530.444	519.222	509.248	500.365	495.171	489.980	484.442	483.354	486.598	484.557	486.023	483.634	479.411	474.213
CHAINAGE	27460	27480	27500	27520	27540	27560	27580	27600	27620	27640	27660	27680	27700	27720	27740	27760	27780	27800	27820	27840	27860	27880	27900	27920	27940

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Title

Geological L-Section
Tunnel No. 8

Scale 0 10 20 30

Date NOV. 2024

Revision 0

Drawing No: RITES/WR/RK/GT/T8

PROPOSED F.L	447.771	447.646	447.521	447.396	447.271	447.146	447.021	446.896	446.771	446.646	446.521	446.396	446.271	446.146	446.021	445.896
GROUND LEVEL	469.123	468.553	467.826	467.746	468.149	468.510	468.725	468.786	468.793	468.744	468.623	468.309	468.359	469.082	469.270	469.030
CHAINAGE	31620	31640	31660	31680	31700	31720	31740	31760	31780	31800	31820	31840	31860	31880	31900	31920

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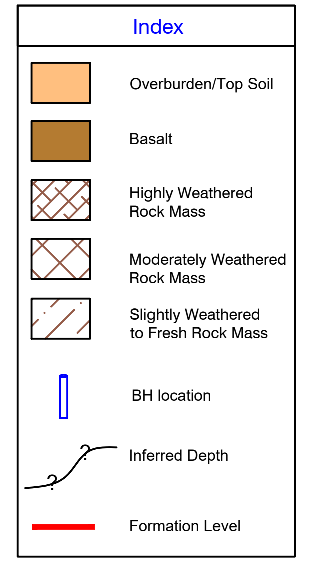
Geological L-Section
Tunnel No. 8A

Scale 0 10 20 30

Date	NOV. 2024	Sheet	1 of 3	Revision 0
Drawing No: RITES/WR/RK/GT/T8A				

PROPOSED F.L	445.896	445.771	445.646	445.521	445.396	445.271	445.146	445.021	444.896	444.771	444.646	444.521	444.396	444.271	444.146	444.021
GROUND LEVEL	469.030	469.527	469.164	468.703	468.003	467.342	466.581	466.502	466.074	465.799	466.157	466.537	466.432	466.334	466.157	465.630
CHAINAGE	31920	31940	31960	31980	32000	32020	32040	32060	32080	32100	32120	32140	32160	32180	32200	32220



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Tunnel No. 8A

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Drawing No: RITES/WR/RK/GT/T8A

PROPOSED F.L	444.021	443.896	443.771	443.646	443.521	443.396	443.271	443.146	443.021	442.896	442.771	442.646	442.521	442.396
GROUND LEVEL	465.630	465.323	465.248	465.045	464.588	464.031	463.479	462.953	462.363	461.747	460.771	459.900	459.075	459.054
CHAINAGE	32220	32240	32260	32280	32300	32320	32340	32360	32380	32400	32420	32440	32460	32480

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Date	NOV. 2024	Sheet	3 of 3	Revision
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PROPOSED F.L	432.896	432.763	432.63	427.192	427.078	426.963	426.849	426.716	426.582	426.449	426.316	426.182	426.049	425.916	425.782	425.649
GROUND LEVEL	450.369	450.808	451.125	454.272	453.954	454.089	454.509	454.460	455.479	456.226	457.209	457.894	458.344	457.682	457.574	457.775
CHAINAGE	34120	34140	34160	35080	35100	35120	35140	35160	35180	35200	35220	35240	35260	35280	35300	35320

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Geological L-Section
Tunnel No. 9

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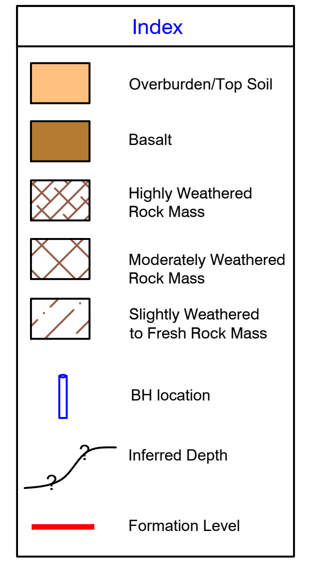
Date	NOV. 2024	Sheet	1 of 2	Revision
Drawing No: RITES/WR/RK/GT/T9				0



PROPOSED F.L	425.649	425.516	425.382	425.249	425.115	424.982	424.849	424.715	424.582	424.449	424.315	424.182	424.049	423.915	423.782	423.682
GROUND LEVEL	457.775	458.280	458.046	457.772	457.010	456.215	455.518	454.837	454.167	453.527	452.971	452.434	451.907	451.020	445.958	434.184
CHAINAGE	35320	35340	35360	35380	35400	35420	35440	35460	35480	35500	35520	35540	35560	35580	35600	35620

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Geological L-Section
Tunnel No. 9

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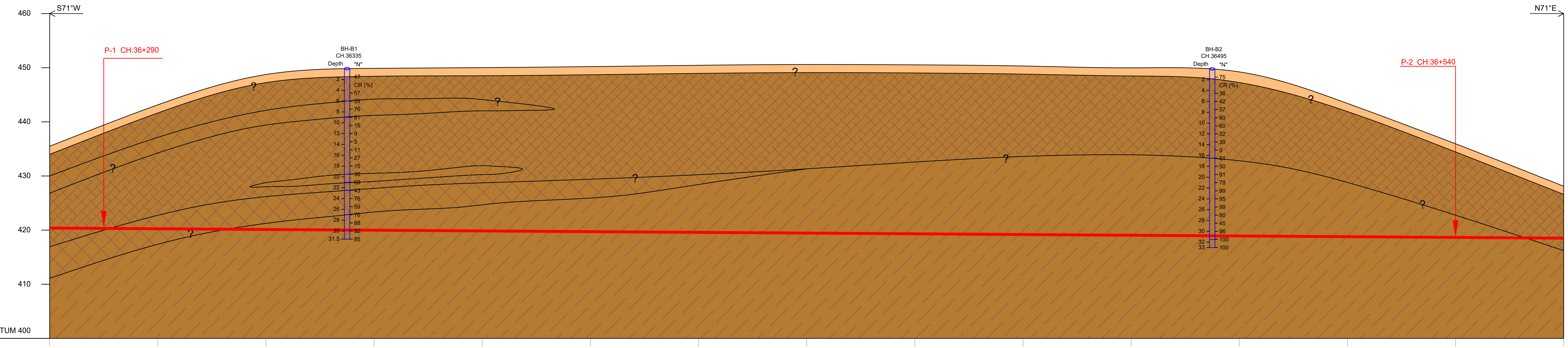

PROPOSED F.L	420.382	420.249	420.115	419.982	419.849	419.715	419.582	419.449	419.315	419.182	419.049	418.915	418.782	418.649	418.515
GROUND LEVEL	435.487	443.142	448.747	449.883	450.016	450.199	450.450	450.594	450.530	450.317	449.976	449.230	443.409	435.946	428.105
CHAINAGE	36280	36300	36320	36340	36360	36380	36400	36420	36440	36460	36480	36500	36520	36540	36560

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Scale 0 10 20 30

Date NOV. 2024

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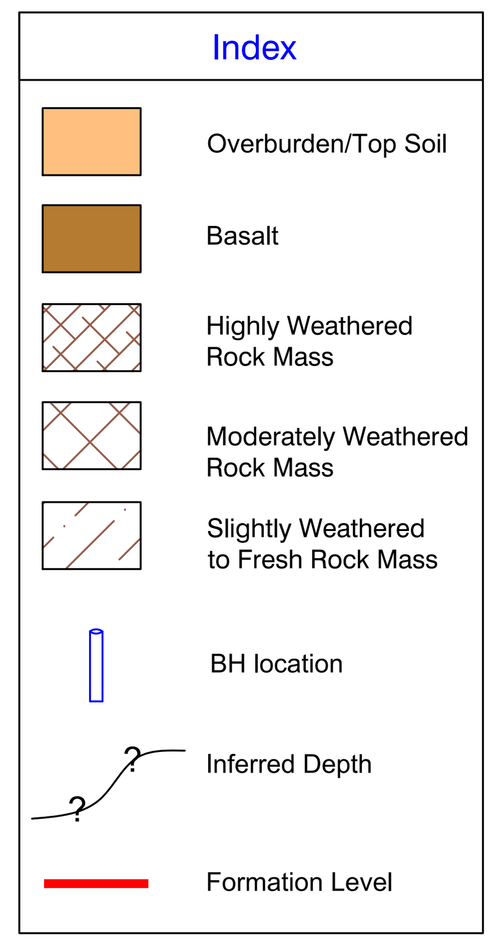
Drawing No: RITES/WR/RK/GT/T9A

PROPOSED F.L	417.282	417.168	417.053	416.939	416.825	416.711	416.596	416.482	416.368	416.253	416.139	416.025	415.911	415.796	415.682	415.568	415.454	415.339	415.225	415.111	414.996	414.882	414.768
GROUND LEVEL	429.050	436.820	444.079	450.390	451.688	452.011	452.395	452.592	452.742	452.688	452.653	452.673	452.283	452.055	451.767	451.457	450.550	449.810	448.840	447.800	440.246	431.202	420.550
CHAINAGE	36800	36820	36840	36860	36880	36900	36920	36940	36960	36980	37000	37020	37040	37060	37080	37100	37120	37140	37160	37180	37200	37220	37240



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Geological L-Section
Tunnel No. 10

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Drawing No: RITES/WR/RK/GT/T10

CHAINAGE	GROUND LEVEL	PROPOSED F.L
37360	424.263	414.082
37380	433.064	413.968
37400	441.031	413.854
37420	446.877	413.739
37440	450.854	413.625
37460	452.065	413.511
37480	452.739	413.396
37500	453.111	413.282
37520	453.505	413.168
37540	453.875	413.054
37560	454.332	412.939
37580	454.385	412.825
37600	454.681	412.711
37620	454.991	412.596
37640	455.455	412.482
37660	456.095	412.368
37680	456.904	412.254
37700	457.510	412.139
37720	458.328	412.025
37740	459.109	411.911
37760	460.388	411.796
37780	463.342	411.682

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Title: Geological L-Section Tunnel No. 11

Scale: 0 10 20 30

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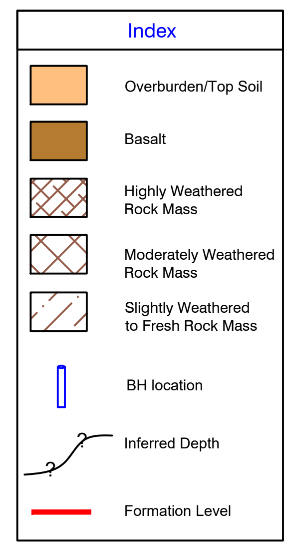
Drawing No: RITES/WR/RK/GT/T11

PROPOSED F.L	411.682	411.568	411.454	411.339	411.225	411.111	410.997	410.882	410.768	410.654	410.539	410.425	410.311	410.197	410.082	409.968	409.854	409.739	409.625	409.518	409.418	409.318
GROUND LEVEL	463.342	468.810	476.105	482.780	486.535	477.653	469.459	463.464	459.902	457.905	456.473	455.551	454.537	452.967	452.009	450.972	450.592	451.628	447.798	439.626	431.325	421.686
CHAINAGE	37780	37800	37820	37840	37860	37880	37900	37920	37940	37960	37980	38000	38020	38040	38060	38080	38100	38120	38140	38160	38180	38200



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Geological L-Section
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Scale 0 10 20 30

Date NOV. 2024	Sheet 2 of 2	Revision 0
Drawing No: RITES/WR/RK/GT/T11		

PROPOSED F.L	407.818	407.718	407.618	407.518	407.418	407.304	407.189	407.075	406.961	406.847	406.732
GROUND LEVEL	422.570	431.687	440.126	447.382	452.757	454.788	455.312	455.980	456.924	457.976	459.460
CHAINAGE	38500	38520	38540	38560	38580	38600	38620	38640	38660	38680	38700

PROPOSED F.L	406.618	406.504	406.389	406.275	406.161	406.047	405.932	405.818	405.704	405.589	405.475
GROUND LEVEL	462.038	464.543	469.368	475.235	484.059	493.184	497.087	497.194	497.259	496.070	494.696
CHAINAGE	38720	38740	38760	38780	38800	38820	38840	38860	38880	38900	38920

PROPOSED F.L	405.361	405.247	405.132	405.018	404.904	404.789	404.675	404.561	404.447	404.332	404.218
GROUND LEVEL	489.936	480.380	472.335	466.521	463.193	461.543	460.026	458.456	459.425	460.306	461.610
CHAINAGE	38940	38960	38980	39000	39020	39040	39060	39080	39100	39120	39140

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Drawing No: RITES/WR/RK/GT/T12

PROPOSED F.L	404.218	404.104	403.989	403.875	403.761	403.647	403.532	403.418	403.304	403.189	403.075
GROUND LEVEL	461.610	463.189	465.345	465.014	464.183	462.543	461.069	458.286	455.821	449.754	445.261
CHAINAGE	39140	39160	39180	39200	39220	39240	39260	39280	39300	39320	39340

PROPOSED F.L	402.961	402.847	402.732	402.618	402.504	402.389	402.275	402.161	402.047	401.932	401.818
GROUND LEVEL	440.550	437.071	433.565	428.190	425.181	425.913	430.037	430.773	435.360	436.035	437.214
CHAINAGE	39360	39380	39400	39420	39440	39460	39480	39500	39520	39540	39560

PROPOSED F.L	401.704	401.589	401.475	401.361	401.247	401.132	401.018	400.904	400.789	400.675	400.561	400.447
GROUND LEVEL	437.994	438.369	436.870	436.703	436.361	435.815	433.247	430.750	426.783	422.661	419.756	416.251
CHAINAGE	39580	39600	39620	39640	39660	39680	39700	39720	39740	39760	39780	39800

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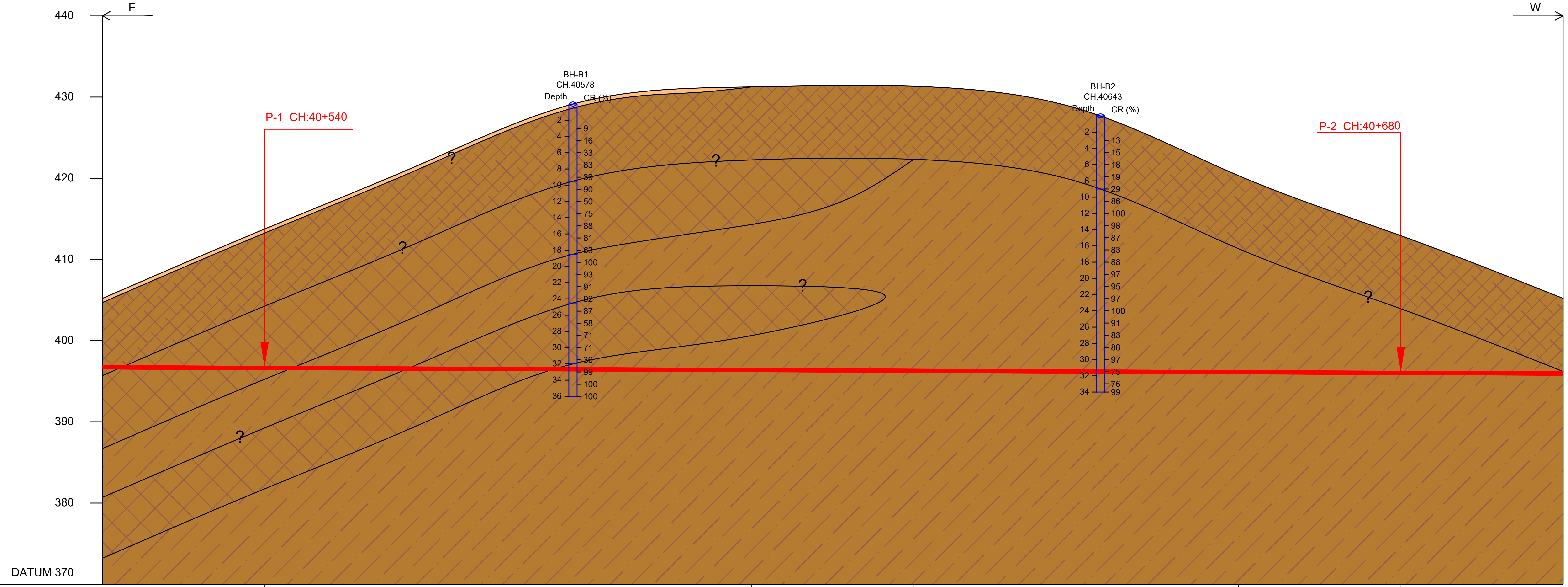

PROPOSED F.L	396.722	396.622	396.522	396.422	396.322	396.222	396.122	396.043	395.993	395.943
GROUND LEVEL	405.188	413.745	422.063	429.633	431.252	431.329	428.639	420.328	412.929	405.204
CHAINAGE	40520	40540	40560	40580	40600	40620	40640	40660	40680	40700

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CHAINAGE	PROPOSED F.L	GROUND LEVEL
42780	388.755	398.563
42800	388.640	407.153
42820	388.525	414.283
42840	388.410	414.995
42860	388.295	415.434
42880	388.180	415.515
42900	388.065	415.588
42920	387.950	415.665
42940	387.835	416.068
42960	387.720	416.609
42980	387.605	416.988
43000	387.490	416.616
43020	387.375	416.494
43040	387.261	415.927
43060	387.146	418.105
43080	387.031	422.431
43100	386.916	430.148
43120	386.792	439.365
43140	386.659	448.661
43160	386.525	450.913
43180	386.392	451.758
43200	386.259	452.426
43220	386.125	453.192
43240	385.992	454.149
43260	385.859	455.065
43280	385.726	456.356
43300	385.592	458.273
43320	385.459	463.010
43340	385.326	470.704
43360	385.192	479.708
43380	385.059	490.203
43400	384.926	501.846
43420	384.792	513.281
43440	384.659	515.452
43460	384.545	510.676
43480	384.430	510.696
43500	384.316	514.271
43520	384.202	516.960
43540	384.088	516.629
43560	383.973	516.002
43580	383.859	517.148
43600	383.745	517.287

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PROPOSED F.L	309.886	309.752	309.619	309.486	309.352	309.219	309.086	308.952	308.819	308.686	308.552	308.419	308.304	308.189	308.074	307.959	307.844	307.729	307.615
GROUND LEVEL	331.735	332.951	333.740	333.901	334.218	334.872	335.189	335.509	335.702	335.686	334.426	337.357	337.364	336.971	339.826	340.392	338.955	339.632	338.118
CHAINAGE	57300	57320	57340	57360	57380	57400	57420	57440	57460	57480	57500	57520	57540	57560	57580	57600	57620	57640	57660

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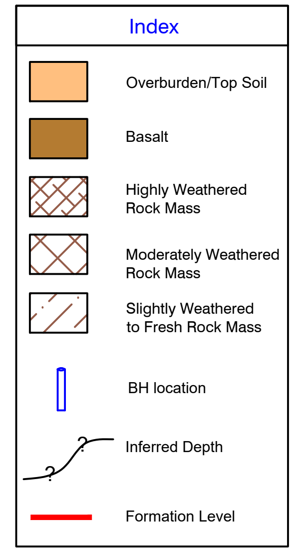
PROPOSED F.L	307.615	307.500	307.385	307.270	307.155	307.040	306.925	306.810	306.695	306.580	306.465	306.350	306.235	306.120	306.006	305.891	305.776	305.661
GROUND LEVEL	338.118	340.902	341.852	342.167	342.391	344.822	347.504	344.666	339.100	334.132	339.943	343.079	343.150	343.165	341.847	328.115	323.057	320.030
CHAINAGE	57660	57680	57700	57720	57740	57760	57780	57800	57820	57840	57860	57880	57900	57920	57940	57960	57980	58000



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Geological L-Section
Tunnel No. 14

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Drawing No: RITES/WR/RK/GT/T14

GEOTECH DIVISION
RITES Bhawan
Shikhar, Sector 29, Gurgaon – 122001 (INDIA)
Email: info@rites.com, Website: www.rites.com

Dr. Ambedkar Nagar to Muktiyara Balwara for Existing/new detour BG line in connection with gauge conversion of existing Ratlam - Khandwa MG section of Western Railway

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TUNNEL-8A (Ch: 31600-32500) m DESIGN DOCUMENT

(January 2024)

A Govt. Of India Enterprise)

RITES BHAWAN

PLOT NO. 1, SECTOR-29

Register of Submissions

Document name:	Tunnel Design Report for Tunnel-8A in Ratlam - Khandwa MG Section

Revision	Date	Description
R-0	06.01.2025	Tunnel Design Report for Tunnel-8A in Ratlam - Khandwa MG Section

Prepared by:	Verified by:	Approved by:
Mr. Rishabh Gupta Mr. Shuvam Bamotra	Dr. Amit Kumar Mrs. Jyotsna Dixit	Mr. R S Dhull

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S. No	Feature	Chainage (Km)	Length of tunnel(m)
1	Portal-1 and adjoining area	31+600	900
2	Portal-2 and adjoining area	32+500	

The proposed alignment of Tunnel T8A between Km 31+600 and Km 32+500 is passing through rocks belonging to Deccan Trap of Upper Cretaceous to Paleocene. Mainly Basaltic Lava Flow (Deccan Trap) is reported all along the tunnel length.

Basaltic Lava Flow (Basalt) is generally fresh to slightly weathered at places moderately weathered, massive, moderately jointed, medium to strong, fine to medium grained, grey to black colour in nature. Normally joint J1 (Primary joint) is horizontal to sub horizontal. Four plus random joints are reported/present in the study area. All the joint sets are almost vertical dipping except J1.

The general trend of rock mass exposed along the entire tunnel alignment in the area. These rock units are traversed by numbers of joint sets where primary/foliation joint is the most prominent one. The joints set in general are close to partly open in nature with medium/moderate to high persistence.

Portal P1 and adjoining area:

The proposed Portal 1 of T8A is located at steep to very steep hill slope ground conditions towards portal and covered with less cover slope wash cum slope debris material & dense vegetation/forest .

Bedrock is not exposed at vicinity of portal area and along the hill slope, but it may be encounter at shallow depth. Overburden materials i.e., slope wash cum slope debris materials is recorded near portal 1 location.

The proposed tunnel portal 1 is having adequate vertical & lateral overburden & suitable for portal location. Seepage is also not observed near proposed tunnel portal.

Figure 1 Showing dense forest cover & hill slope debris materials

The upslope area has also occupied by thick forest cover/vegetation and having steep towards portal. The portal slope area of proposed portal is covered with thin slope debris cum hill slope wash material represented by pebbles to boulders of Basalt embedded in silty clay matrix. The overburden material is unconsolidated to semi-consolidated in nature consisting of pebbles, cobbles, boulders of Basalt and other rock fragment mixed with soil matrix. The bedrock is not recorded at portal location, but it may be encounter at shallow depth.

<u>Section Between Portal P1/T8A and Portal P2/ T8A:</u>

The area between Portal 1 to Portal 2 of proposed T8 tunnel alignment passes through hill with moderate to steep slope and area partly covered with slope wash cum slope debris material & thick vegetation/ dense forest followed by compact, massive, hard, medium strong to strong, thickly spaced, grey to black colored, fine to medium grained, fresh to slightly weathered, Basalt . Maximum part of the corridor is covered with less/thin overburden material which is unconsolidated to semi consolidated in nature and the visually estimated ratio between the coarse: fine fraction is 90:10 respectively. Seepage & nala is not observed between proposed tunnel portal 1 & portal 2.

Figure 2 Showing dense forest cover, hill slope debris & minor rock exposure.

The general trend of rock mass exposed along the entire tunnel alignment in the area. These rock units are traversed by joint sets where foliation joint is considered as most prominent joint set. The joint sets in general are close/tight to partly open in nature with high persistence.

2 <u>Alignment Details</u>

The provided tunnel alignment in by RITES is tabulated as below:

Table 1 Proposed alignment T-8A

S.No.	Parameter	Proposed alignment
1	Tunnel length	900 m
2	Start chainage	km 31+600
3	End Chainage	km 32+500
4	Straight/ Curved tunnel alignment	Curved
5	Maximum overburden (from FL)	23 m
6	Maximum gradient	Fall 1in160

The details of the proposed tunnel alignment with its geometric properties are as summarized table below-

Table 2 Geometric details of tunnel T-8A alignment

S.No.	Curve radius	Points on curve	Chainage
1	1750 m (Curve no.19R)	TPTC-1	km 31+616.399
		TPCC-1	km 31+726.399
		TPCC-2	km 32+600.500
		TPTC-2	km 32+710.500

3 Theory on Primary lining

3.1 Primary Support

The purpose of the primary support is to stabilize the underground opening until the final lining is installed. In many cases it may become necessary to apply the support system in combination with auxiliary constructional measures. The most common elements for the primary support are:

a) Rock bolts: Rock reinforcement in tunnels (of which rock bolt is one of the types) is used for many purposes. There are many types of rock bolts, with many of them being patented products, generally made of steel (mostly reinforcement bars of suitable diameter).

A borehole of required diameter is drilled in the rock mass and the bolt is inserted in the borehole. The bolt is anchored near the tip using suitable mechanism, then it is stressed, and the bore hole mouth is covered by using a face plate and face nut. Afterwards, grouting is done (in most of the cases) to fill up the annular space between the bolt and the walls of the borehole. In case bore hole does not remain stable until withdrawal of drill rod and insertion of bolt, Self-Drilling Rock bolts (SDR) or Self-Drilling Anchors (SDA) are used, wherein the drill bit is located at the end of bolt and borehole is drilled using the bolt and drill bit. In such a system, the drill bit is not re-used, and one drill bit gets scarified in each borehole. SDAs or SDRs are costly as compared to normal rock bolts, but their use is necessary in case of weak ground tunnelling. When the required length of rock bolts becomes excessive (say more than 10m or so), rock anchors (which are made of woven high tensile steel ropes) are used in pace of steel bolts.

Rock bolts serve many purposes in the tunnels. Major purpose of rock bolts is to support individual rock block(s), which may become loose due to creation of cavity and may have eventually fallen in figure below.

b) Shotcrete: Shotcrete is the process in which cement, sand and fine aggregate concrete conveyed through a hose and pneumatically projected at high velocity onto a surface, as a construction technique.

Dry Mix Shotcrete: Dry shotcrete components – which may be slightly pre-dampened to reduce dust – are fed into a hopper with continuous agitation.

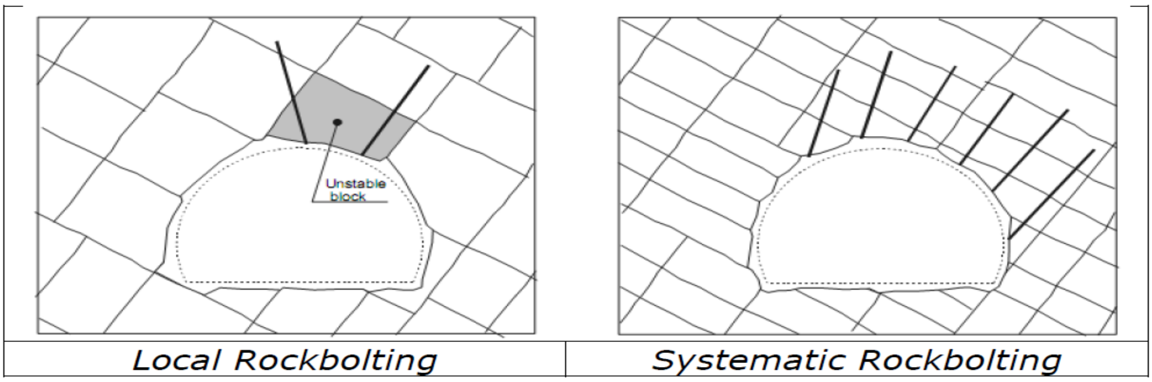
Compressed air is introduced through a rotating barrel or feed bowl to convey the materials in a continuous stream through the delivery hose. Water is added to the mix at the nozzle. Gunite, a proprietary name for dry- sprayed mortar used in the early 1900s, has fallen into disuse in favour of the more general term shotcrete.

Wet Mix Shotcrete

In this case, shotcrete components and water are mixed (usually in a truck-mounted mixer) before delivery into a positive displacement pumping unit, which then delivers the mix hydraulically to the nozzle where air is added to project the material onto the rock surface. The final product of either the dry or wet shotcrete process is very similar. The dry mix system tends to be more widely used in tunnelling because of inaccessibility for large transit mix trucks and because it generally uses smaller and more compact equipment. This can be moved around relatively easily in an underground environment. The wet mix system is ideal for high production applications in tunnelling and civil engineering where a deep shaft or long tunnel is being driven and where access allows the application equipment and delivery trucks to operate on a relatively continuous basis. Decisions to use dry or wet mix shotcrete processes are usually made on a site-by-site basis.

c) **Steel ribs and lattice girder:** They are fabricated at site, to the required cross section of tunnel, by welding of reinforcement bars. These types of supports are relatively light weight (hence economical), flexible and easy to handle. They are normally installed with shotcrete (Fig. below). Use of such supports is very common nowadays and their flexibility is an added advantage, as will be discussed subsequently in “Rock Structure Interaction”.

Rock bolts



Rockbolting

Figure 3 Rock bolting

Figure 4 Shotcrete, steel ribs Lattice Girder

This report covers analysis and primary design of tunnel T-8A in Ratlam-Khandwa section. Analysis of excavation and support for tunnel has been carried out for different ground type which are anticipated to be encountered i.e., Grade-IV & Grade-V based on Q classification.

4 References

I. Austrian Society for Geomechanics: Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation, 2010

II. Practical Rock Engineering by Dr. Evert Hoek, 2004

III. Austrian Society for Geomechanics: Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation.

IV. Austrian Concrete Society Publications, Guideline Sprayed Concrete, 2013.

5 Geotechnical Design parameters

Geological survey and mapping of the project area indicate that the tunnels are to be excavated in Granitic Gneiss rock comprising majorly Jointed strata. The discontinuities intersecting the rock mass are related to low tectonic history and mechanical properties of individual rock types. On the scale of tunnel profile, the occurrence and frequency of discontinuity sets is variable.

The representative rock mass conditions/ ground types were identified in terms of Geological Strength Index (GSI) values which have been directly taken from the aid of standard chart applicable for the jointed rock mass by P. Marinos and E. Hoek (2000).

Various ground types are classified in rock grades viz. Grade I, Grade II, Grade III, Graded IV and Grade V for evaluating the rock mass parameters, based on observed rock mass conditions, out of which Grade III & V are observed in tunnel.

For each Ground type, the referenced GSI value has also been categorized into a high and low value to account for the minor reduction in quality of joint conditions to differentiate between the weathering conditions. The range of intact rock strength (σci), GSI, intact modulus (Ei) and corresponding rock mass modulus (Erm) values are as per GIR report of tunnels (submitted separately).

Tunnel is divided into sections based on the observed rock quality. The rock mass properties for each section are further assessed for the appropriate rock mass overburden observed in those sections.

It is further observed that there is persistent ground water present across the tunnel. Seasonal dripping of water could also be observed at some places which is taken in analysis.

Considering a conservative case to estimate maximum stresses on lining in the conditions observed at site a value of 0.5 is selected for K_0 (gravity field stress ratio). According to geological and geotechnical conditions, the geotechnical design parameters associated to the different ground types for each tunnel are defined and given in tables below. These geotechnical parameters associated to the different ground types were considered for the analyses.

Table 3 Geotechnical Design Parameter for Tunnel-8A

GEOTECHNICAL PARAMETERS USED FOR DESIGN ANALYSIS FOR TUNNEL DESIGN									
	Chainage	**Depth of Borehole**	**Borehole**	**Tunnel**	**Poissons Ratio (ʋ)**	**Angle of internal friction (ø)**	**Cohesion (c)**	**Young Modulus (E)**	**Unit Wt.**
UNITS	**Km**	**m**	**Type**	**Type**	**-**	**-**	**Mpa**	**Mpa**	**kN/m³**
TUNNEL DESIGN PARAMETERS	31+640	30.0	BH-1	T8A	0.22	48.903	0.264	951.152	2.3
	32+460	25.0	BH-2		0.20	43.708	0.100	200.709	1.9

The Hoek-Brown Failure criteria is used in analysis of rock mass parameters from intact rock parameters. ***Details have been mentioned below:***

5.1 HOEK-BROWN FAILURE CRITERION

The Hoek-Brown failure criteria is a widely used rock mass failure criterion that is based on empirical observations of the behaviour of rocks under stress. This criterion was first proposed by Hoek and Brown in 1980 and has since become an important tool for predicting the behaviour of rock masses in engineering and mining applications.

The Hoek-Brown failure criterion involves a total of 8 parameters: the uniaxial compressive strength of the rock (σ_{ci}), Intact rock parameter (m_i), Geological Strength Index (GSI), Young's modulus of rock mass (E), Poisson's ratio (μ), Disturbance factor (D), Drained cohesion (c'), Drained friction angle (φ), Absolute value of confining pressure σ'_3 at which $\psi = 0°$, Dilatancy angle (ψ_{max}) at $\sigma'_3 = 0$.

The Hoek-Brown failure criterion is particularly useful because it takes into account the effects of confinement and the brittleness of the rock mass. It has been shown to provide more accurate predictions of the behavior of rock masses under stress than other failure criteria, particularly in situations where the rock mass is heavily jointed or fractured.

The generalised Hoek-Brown failure criterion can be expressed as a non-linear relationship between the major and minor effective principal stresses, as shown below.

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

Where,

σ_1' and σ_3' are the major and minor effective principal stresses at failure;

σ_{ci} is the uniaxial compressive strength of the intact rock material and

m_b is a reduced value of the intact rock parameter m_i as given by;

$$m_b = m_i\, exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

s and a are auxiliary material constants for the rock mass given by the following relationships;

$s = exp\left(\frac{GSI-100}{9-3D}\right)$ and $a = \frac{1}{2} + \frac{1}{6}\left[exp\left(\frac{-GSI}{15}\right) - exp\left(\frac{-20}{3}\right)\right]$

The Roc Data results are placed on Appendix-I , whereas the theory for Geological Strength Index (GSI), Disturbance factor (D) and Intact rock parameter (m_i) are estimated based on Hoek & Brown (2002) given in Appendix II of this report.

D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. The detailed report of interpretation of intact rock property into rock mass is tabulated in Annexure-1.

6 <u>Design Methodology</u>

The primary purpose of any tunnel site investigation i.e., Geo-mapping, Geophysical survey and Geotechnical investigations is to obtain the maximum amount of information on rock characteristics, structural systems, and groundwater conditions. This information is important for as it enables the designer to assess the behaviour of the rock surrounding the tunnel and the type of support required to maintain the tunnel in a stable condition. With the designed rock supports the contractor can plan and schedule the construction activities to complete the work

Rock mass classification schemes were developed for over 100 years with an attempt to formalize an empirical approach to tunnel design, in particular for determining support requirements. While the classification schemes are appropriate for their original application, especially if used within the bounds of the case histories from which they were developed, considerable caution must be exercised in applying rock mass classifications to other rock engineering problems. The RMR by Bieniawski's and Q-value by Barton et al' are most commonly used in tunnel design.

RMR System

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (RMR) system. The following six parameters are used to classify a rock mass using the RMR system:

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

The RMR value of rock mass is the summation of all these six parameters. The Rock Quality is based on the RMR value of rock mass and based on the Rock Quality support system is proposed by the Bieniawski. The relation between RMR rating and rock Quality is given below in table.

Table 4 Relation between RMR and Rock Quality

RMR (Rock Mass Rating)	Rock Quality
81-100	Excellent
61-80	Good
41-60	Fair
21-40	Poor
0-20	Very poor

The Guidelines for excavation and support of 10 m span rock tunnels in accordance with RMR system is given in table below.

Table 5 Support based on RMR Value

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock RMR: 81-100	Full face, 3 m advance.	Generally, no support required except spot bolting.		
II - Good rock RMR: 61-80	Full face, 1.5-3.0 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.

III - Fair rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock RMR: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Q- System

Based on an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{Jn} X \frac{Jr}{Ja} X \frac{Jw}{SRF}$$

Where,

RQD is the Rock Quality Designation

J_n is the Joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor

Estimated support categories based on the tunnelling quality index Q after Grimstad and Barton, 1993, reproduced from Palmstrom and Broch in 2006. The Chart representing the rock type and support proposed based on the Q-value is given in the Figure below.

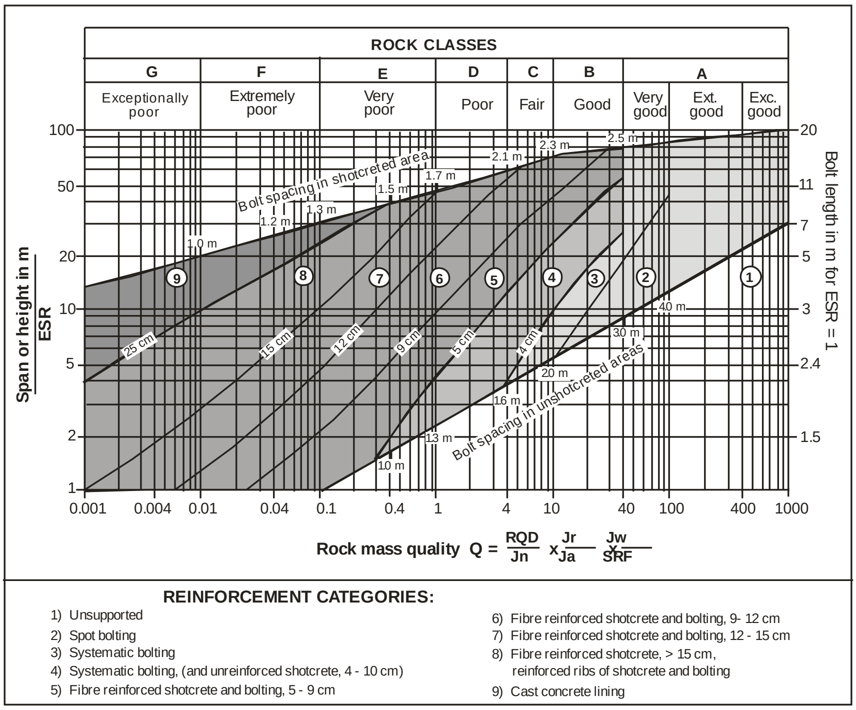


Figure 5 Showing the Rock Type based on Q value.

Thus, following values are interpreted based on considering RMR & Q for classification of Tunnel section.

Table 6 Support Class Recommendation For Tunnel Primary Lining

Support Class	Support Class Listing	Modified Support Class on account of RMR & Q classification	Advance length	Bolt Length (m)	Spacing(m)	M 25 Shot crete (cm)	Wire Mesh (150 x 150 x 6)	Lattice Girder 70/16/25 @ 11.5 kg/m	Fore poling 25 mm dia
1	Unsupported	I	Full face, 3 m advance.	NA	NA	NA	NA	NA	NA
2	Spot Bolting			NA	NA	NA	NA	NA	NA
3	Systematic Bolting	II	Full face, 2m advance. Complete support 20 m from face.	3	2.0	5 cm	NA	NA	NA
4	Systematic Bolting and unreinforced shotcrete 4-10 cm			3	2.0	5 cm	NA	NA	NA

5	Fibre Reinforced Shotcrete and bolting 5-9 cm	III	Full face, 2m advance. Complete support 10 m from face. Commence support after each blast.	4	2.0	10 cm	Yes	NA	NA
6	Fibre Reinforced Shotcrete and bolting 9-12 cm			4	2.0	10 cm	Yes	NA	NA
7	Fibre Reinforced Shotcrete and bolting 12-15 cm	IV	Full face, 1.5 m advance. Install support concurrently with excavation, 10 m from face.	4	1.5	15 cm	Yes	Yes	NA
8	Fibre Reinforced Shotcrete > 15 cm reinforced ribs (70/16/25 @ 11.5 kg/m) of shotcrete and bolting			4	1.5	15 cm	Yes	Yes	NA
9	Cast concrete lining	V	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	6	1.5	20 cm	Yes	Yes	Yes

Following loads are considered for the primary support design in the numerical calculations:

6.1 Earth Pressure

Ground loads on initial stress state given by a maximum overburden according to cross section considered and a coefficient of earth pressure at rest of 0.5 is considered. Ground loads onto the tunnel cross section are calculated by definition of a relaxation factor, which simulates the “volume loss” and stress relaxation due to excavation, followed by a complete relaxation of the ground inside of the excavation area. The earth pressure for the analysis is derived in consideration of overburden and unit weight of rock mass.

6.2 Groundwater pressure

During geotechnical investigations and site visit, it was observed that water in tunnels was limited to dripping conditions (with seasonal occurrences). As the strata is competent, water pressures are not expected to develop beyond tunnel linings. Hence, the tunnels are designed as drained tunnels, for which water pressures are not considered as design load case. Additionally, drainage path with use of waterproofing membrane and drains will ensure that no water pressure builds up on tunnel linings.

6.3 Live load

Any live loads if applicable are considered in the design for the design of primary lining. For these tunnels there are no live loads expected.

6.4 Seismic Load

With regard to the temporary works character of the outer lining (primary support) seismic loading is not applicable here. Whereas the same is considered in permanent lining design.

7 Safety Factors

For all loads relevant to the primary support, a load factor of 1.0 is considered due to the temporary nature of the primary structures.

For design of the primary (temporary) support, the following material factors have been applied:

Table 7

Steel (reinforcement, rock bolts etc.):	1.15(applied to yield strength)
Shotcrete:	1.50

8 Proposed support systems

The following properties of rock support elements are considered for the primary support design:

8.1 Shotcrete

Specified compressive strength after 28 days is corresponding to concrete grade M25, characteristic compressive strength fck = 25 N/mm2, according to IS 456:2000.

Young’s modulus:	E = 25000 MPa
Poisson’s ratio:	ν = 0.2
Unit weight:	γ = 24 kN/m3

8.2 Reinforcement Steel

Steel reinforcement for shotcrete: Wire mesh (welded steel wire fabric): Grade Fe 500D, according to IS 1786:2008

Mesh width: 150 mm x 150 mm; bar diameter – 6mm (according to IS 4948)

Minimum yield strength: fy =500.0 N/mm2

8.3 Rock Bolts

Fully grouted rock bolts such as SN bolts or self-drilling (SD) bolts are used. SN bolts are considered with tensile capacities as mentioned in drawings and specifications.

8.4 Lattice girders

Lattice girders used in the design are 70/16/25 type three legged systems.

8.5 Fore poling.

Fore poling for the tunnels is proposed with 4m length 25 mm dia bars (min. Fy = 210 kN/mm2) having 1m overlap, 300-400 mm c/c distance between bars and 110^{o}coverage at crown.

8.6 Wire mesh with clamps

Wire mesh (hexagonally twisted) with Zinc coating and clams are provided to bottom two slope to avoid any rock fall on the track. This will arrest any such situation due to natural weathering of rock surface during design life of the project.

9 Calculations

9.1 Results and support summary

The results of the empirical analyses are given in below. Five support classes have been designed corresponding to all anticipated ground types for each tunnel.

Tables below shows the basis of calculation of Q value based on three approaches used for geotechnical assessment,

a) Geomapping
b) Drilling data bore logs.

Based on the above-mentioned approaches, support for each tunnel section chainage wise is derived and the adopted value based on average/ most appropriate of these values is concluded.

Tunnel T-8A is 900 m in length. The tunnel has been divided into two sections based on above mentioned parameters. For global stability of portals support class IV of 70 m from Portal-1 ie. 31600 and 100 m from Portal-2 ie. 32500 is proposed at each portal location. On interpolation of all results following is concluded. Similar quantities are computed in Bill of Quantities. It may please be noted that Kinematic analysis of Tunnel-8A is not performed as rock exposures at tunnel portals and intermediate chainage is not fully exposed and the data is not sufficient for analysis and during excavation the geologist engaged may be required to provide adequate measures considering the site condition. However, as per geology and other governing parameters the results and corresponding quantities are considered for present condition.

S.NO.	TUNNEL TYPE	Governing Bore Hole	GEOMAPPING RESULTS	DRILLING RESULTS	Adopted Results	Chainage from	Chainage to	Remarks /Support Type	
Unit	T = TUNNEL	BH No.	Q RATING	Q RATING	Q Rating	m	m	Type	Support
1	TUNNEL-8A	BH-1	1.0-4.0	1.0-40.0	1.0-4.0	31600	31670	Slope Debris with patches of slightly weathered, Basaltic Lava Flow (Basalt) along the profile	SC-IV
		BH-1 & BH-2	4.0-40.0	4.0-10.0	1.0-40.0	31670	31960		SC-III
		BH-1 & BH-2	4.0-40.0	4.0-10.0	1.0-40.0	31960	32110		SC-II
		BH-1 & BH-2	4.0-40.0	4.0-10.0	1.0-40.0	32110	32400		SC-III
		BH-2	1.0-4.0	4.0-10.0	4.0-10.0	32400	32500		SC-IV

Table 8 PRIMARY LINING OF TUNNEL -8A BASED ON Q SYSTEM

10 Secondary Lining Approach and Methodology

The initial shotcrete lining for a NATM tunnel is designed and installed as temporary support for carrying ground loads induced by tunnel excavation. The initial shotcrete lining may degrade over time, particularly in aggressive environments due to the corrosion of steel reinforcement subject to high chloride conditions or deterioration of the shotcrete because of the presence of sulphates in the ground or groundwater. This degradation of the initial shotcrete lining causes redistributions of stresses and strains or loads in the lining and adjacent ground, and possibly additional deformation of the lining. As a result of the degradation and additional deformation of the initial lining, the loads originally developed in the initial lining will redistribute to both the adjacent ground and secondary lining over the long term.

The magnitude of load transferred from the initial lining to the secondary lining, also called the load sharing, depends on many factors, including:

- Available bond and normal and shear stiffness of the interface
- Ground conditions such as rock mass strength/stiffness
- Relative stiffness between the initial and secondary linings
- In situ stress conditions such as the ratio of horizontal-to-vertical stresses (Ko)
- Tunnel shape (e.g., circular and horseshoe shaped).

The secondary lining extrados will be in contact with the waterproofing membrane and potential groundwater in the event of a breach of the waterproofing membrane. The intrados is exposed to the internal atmospheric conditions of the individual tunnel cross passage. The primary deterioration mechanism which risks secondary lining durability is carbonation. Secondary lining extrados is not accessible for visual inspection and/or repair while the intrados will be accessible for inspection, monitoring, and maintenance but only during limited planned engineering possessions of the tunnel.

The sprayed concrete primary lining is a temporary element which is to stabilise the ground during the construction period, providing support until the permanent secondary lining is installed. The primary lining has no permanent support function.

The secondary tunnel lining is designed after considering the factors like the deformation modulus, UCS, loads acting on an approach that enables the specific characteristics of a tunnel lining implemented using numerical modelling to be considered during its design.

11 Design loads and safety factors

Following loads are considered for the primary support design in the numerical calculations:

11.1 Earth Pressure

Ground loads on initial stress state given by a maximum overburden according to cross section considered and a coefficient of earth pressure at rest of 0.5 is considered. The earth pressure for the analysis is derived in consideration of overburden and unit weight of rock mass.

11.2 Live loads

Any live loads if applicable are considered in the design for the design of primary lining. For these tunnels there are no live loads expected.

11.3 Seismic loading

The project area lies in Zone 2 as per IS 1893-I (2002). The earthquake loading is applied as per the code and vertical seismic coefficient used for analysis is 0.06 and horizontal seismic coefficient used in 0.04.

11.4 Groundwater pressure

For “drained” tunnels, water pressures are not considered as design load case as the water in tunnels is observe in dripping conditions only with seasonal appearance. Additionally, drainage system in tunnel shall ensure that no water pressure builds up.

11.5 Safety Factors

For all loads relevant to the secondary support, a load factor of 1.3 will be considered due to the permanent nature of the secondary lining.

For design of the secondary (permanent) support, the following material factors have been applied:

Table 9

Steel (reinforcement etc.)	1.15(applied to yield strength)
Concrete:	1.50

12 Proposed support systems

The following properties of rock support elements are considered for the secondary support design:

12.1 PCC lining

The stretches of tunnels which are in good rock conditions were analyzed for stresses expected on lining and PCC lining is proposed where there are no stresses expected and rock mass undergoes full relaxation during application of primary support. The compressive strength of PCC lining proposed is M30 after 28 days, characteristic compressive strength fck = 30 N/mm2, according to IS 456:2000.

For Grade III rock, PCC Lining was modelled in RS2 software for the worst Geotechnical Parameters and the resultant forces & Moments are checked for the adequacy of the PCC lining. The Model is attached in this report below.

12.2 RCC lining

RCC lining is proposed for Support Class -IV of tunnel from both ends i.e., tunnel portals to provide stiffer openings to the tunnel entrance and to cater the possibility of finding weathered rock at low overburden region. Additionally, where the tunnels are going through weaker rock strata and ground has capacity to relax in long term the RCC lining is proposed. The steel used in Fe 500/Fe500D as per IS 1786: 2008.

For Grade V rock (weaker and weathered rock types), RCC Lining was proposed and modelled in RS2 software for the worst geotechnical parameters encountered in the project and the resultant forces & moments are checked for the adequacy of the RCC lining. The Model is attached in this report.

13 Calculations

13.1 Numerical Modelling

The software used for the numerical analysis is the two-dimensional finite element software RS2 version 9 from Ro science. This software is intended for 2D elasto-plastic finite element stress analysis for underground excavations in rock or soil.

13.1.1 Material models

Material models are used to describe the behaviour of the ground that is suitable constitutive laws to account for the elastic, as well as inelastic ranges of the respective materials.

The material behaviour of the ground is simulated according to the material law of Mohr-Coulomb in this project.

13.1.2 Ground Loads – Representation of the Construction Sequence

Tunnel excavation causes a disturbance of the initial stress state in the ground and creates a three-dimensional stress regime in the form of a bulb (arching effect) around the advancing tunnel face.

The extent of the stress disturbance around an active heading depends mainly on ground conditions, size of the excavation and length of round. The design according to NATM principles dictates limits on excavation size and length of round and prescribes installation of primary support elements immediately after the excavation of each individual round. Primary support elements are therefore installed within the region of a load-carrying arch around the newly created opening in the region where some pre-deformation has occurred.

As the excavation of the tunnel advances the shotcrete hardens from an initially "green" shotcrete and becomes fully loaded at a certain distance from the face. Such sequencing combined with the early support installation contributes to the development of the self-supporting capability of the ground. It further helps in minimizing deformations and ground loosening. It is therefore important to portray the excavation and support sequencing closely in the numerical analyses.

Further, to design the secondary lining, it is assumed that the primary lining does not exist, and the stresses post relaxation of rock are taken by the secondary lining itself.
Details of ground type with corresponding support class have been summarized in table given below.

Table 10 Ground type and associated excavation sequences

Ground Type	Support class	Excavation method
Grade I	As specified in the drawings	Full face
Grade II	As specified in the drawings	Full face
Grade III	As specified in the drawings	Full face
Grade IV	As specified in the drawings	Full face / Heading & Benching
Grade V	As specified in the drawings	Heading and Benching

Table 11 Calculation stages for numerical analyses

Stage No.	Description of calculation stages
1	Grid set up and initial stress field
2	Face Excavation
3	Application of Primary Support Shotcrete, wire mesh, lattice girder rock bolt (only if the deformation in the rock pass is resulting in plasticization of ground near the tunnel)
4	Deactivation of primary support and application of secondary support.

13.1.3 Limit state analysis

The analysis for secondary lining is performed for limit state with required factor of safety of 1.5.

13.1.4 Lining Forces

From the numerical analyses, sectional forces (axial forces and shear forces) and bending moments of the lining are evaluated. The combinations of these sectional forces and bending moments are used to evaluate the capacity of the lining.

Based on this evaluation, the adequacy of the lining thickness and its reinforcement (if any) is assessed. The lining will be reinforced as required by the analysis.

The structural design is carried out in accordance with IS 456:2000 Plain and Reinforced Concrete Code of Practice and EN 1992-2005.

Partial safety factors for materials for ultimate limit states are adopted according to Indian codes IS 456- 2000.

Table 12 Partial factors for materials for ULS

Load Combination	Concrete	Reinforcement Steel
Ordinary Load Combination	1.5	1.15

The minimum concrete covers to all reinforcement (main and distribution reinforcing bars) considering the exposure conditions are adopted as follows:

- Concrete exposed to earth (external face) 50 mm /75 mm for foundation.
- Concrete not exposed to earth (internal face) 50 mm.

14 Results

14.1.1 Roc Support

Given the tunnel radius, in-situ stress conditions, rock parameters and support parameters, a ground reaction curve and a support reaction curve are calculated. The intersection of these curves determines a factor of safety for the support system. Account for long term tunnel behaviour by applying a strength reduction factor to obtain a long-term Ground Reaction Curve.

For Grade IV and Grade V, at maximum overburden depth and tunnel portals, the analysis has been done and the results are shown below, Factor of safety of 3.56 is achieved with given parameters.

(a) Max. Ht at tunnel height of 40 m

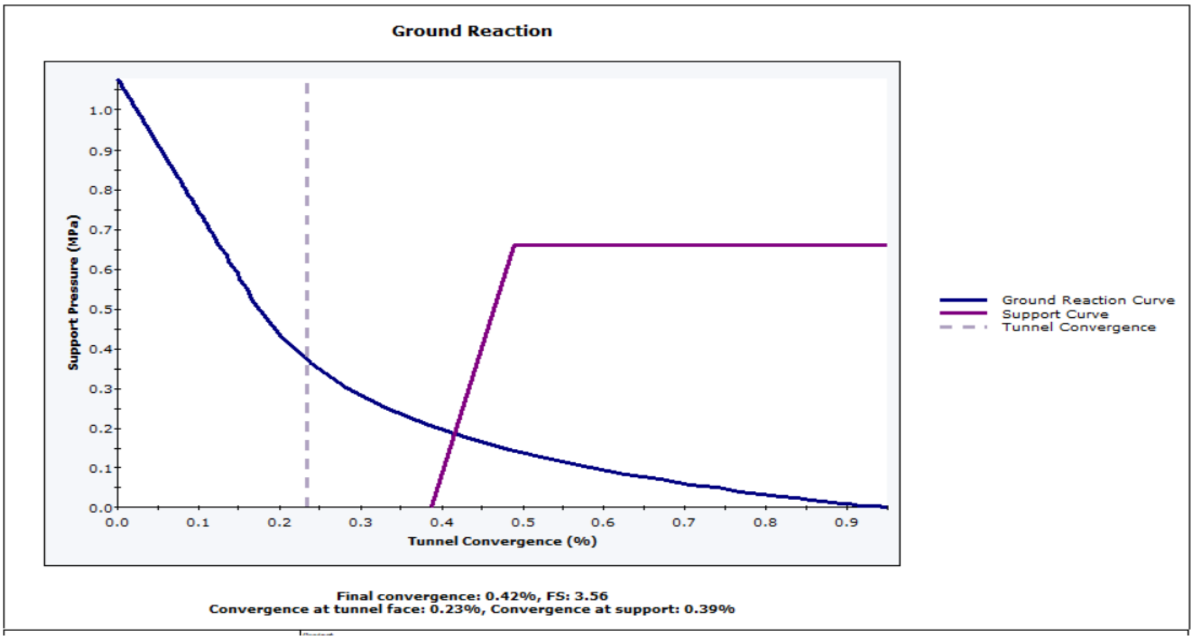


Figure 6 Roc Support Software for Grade IV

15 Portal Slopes Design basis and working

15.1 RDSO Standards for portal design

The evaluation of portal locations for mountain tunnels is among the most crucial considerations during route selection and structural layout planning. The development of spatial information technology has provided a more objective approach for assessing the slope stability of potential portal sites. An empirical method suggested by RDSO, for an infinite slope was integrated into the geographical information system for evaluating the stability of critical wedges. The proposed method provides a reasonable estimation comparable with that provided by the stability software's. The results of applying this method to tunnel portals where slope instability is significant. For potential portal site evaluation, the proposed method facilitates the rapid estimation of safety factors for various slope designations, which is useful for site selection. Tunnel proposed at Ratlam-Khandwa section; therefore, number of portals (one at each end) need to be analysed for structure stability.

Slope failure of cuttings is a complex phenomenon. The failure of rock slopes is controlled by geological features such as bedding planes and joints which divide the rock body up into a discontinuous mass. Under these conditions, the failure path in rocks is normally defined by one or more of the discontinuities. However, in the case of soil, a strongly defined structural pattern does not exist and therefore, the failure surface is free to find the path of least resistance through the slope.

16 Slope Stability

The slope stability is discussed for soils and rocks separately.

16.1 Soils

Failure in soils is rather simple i.e., circular in homogeneous materials & non-circular or planar in layered soils. In general, failure along a non-circular surface can be anticipated if the soil deposit is non-homogeneous or if there are discontinuities within the slope. A predominantly planar slip surface may be expected in shallow natural slopes. Failure generally takes place along slip surfaces parallel to the slope.

The stability of any slope is governed by several factors such as the nature of materials comprising the slope, the history of slope formation, the movement of water through the soil and the steepness of the slope. The most common type of failure in soil is that due to sliding and it is often referred to as a shear failure along a surface of sliding. The tendency for instability or failure is a consequence of gravity (self-weight of the soil or soils comprising the slope) and any other external loads (e.g., a structure on the crest of the slope, water pressure in tension cracks or an earth tremor). This tendency is resisted by the shear strength of the soil or soils comprising the slope. In a stable slope there is no continuous surface along which the average shear strength is less than the average shear stress caused by gravitational and external loads. Zones of overstress could be the starting points for local, partial, or complete failure. The formation and propagation of such zones is sometimes crucial for the safety of a soil mass. To understand the conditions governing stability fully, it is useful to consider geological, geotechnical, and environmental factors.

The soil adjacent to the ground and slope surfaces may be quite strong but there may be a bedding plane or a fault or an ancient surface of sliding within the slope. An understanding of local geology facilitates possible detection of such features at a given site.

The shear strength that can be mobilized is governed by the permeability characteristics and the extent of drainage and volume change that can take place. Such geotechnical factors require careful attention. Infiltration of water due to rainfall increases pore water pressure and reduces the shear strength. Often slope failures occur because of heavy or prolonged rainfall.

Environmental changes near a sloping area such as deforestation, urbanization and construction of reservoirs often lead to increases in pore water pressure and other effects such as soil erosion. Filling of valleys may also disturb the natural drainage characteristics of a sloping area and contribute to instability.

It is further observed that there is no persistent ground water present across the tunnels. Seasonal dripping of water was observed at some places which is ignored for analysis purposes. K_0 (gravity field stress ratio) is considered as 0.5 for analysis purposes. Distinction must be made between short-term and long-term stability conditions, especially for slopes of cohesive soil. In the field, the end of construction situation is usually a short-term stability condition. The long-term condition is when ‘excess’ or ‘transient’ pore water pressures within a slope are fully dissipated. However, the long-term condition of equilibrium may be reached in the field after many months or years depending on the thickness of cohesive soil, its coefficient of permeability and other factors. For cuts, excavations and natural slopes, critical stability is in the long term when the factor of safety is a minimum.

Unloading or excavation causes negative excess pore water pressure. Consequently, the total pore water pressure has its lowest value at the end of construction and shear strength has its highest value at the time. In the long term, the negative excess pore water pressure reduces to zero, the total pore water pressure is increased, and the shear strength is, therefore, decreased. It is obvious that the stability of an excavation is reduced in the long term from the condition at the end of construction.

The complete dissipation of excess pore water pressures in a cohesive soil may take many years. In cohesionless soils like sand, excess pore water pressures are dissipated so rapidly that there is no need to distinguish between short-term and long-term conditions based on pore water pressure and drainage. However, this is true of static loading only. Significant excess pore water pressures may develop in such soils during earthquakes resulting in dramatic loss of shear strength.

16.2 Rocks

The properties of intact rock are changed dramatically by the presence of discontinuities such as joints, faults, and fractures. These discontinuities are planes of weakness across which there is little or no tensile strength. In essence, discontinuities break the cohesive bonds across distinct planes in the rock. On a local scale, this may cause the tensile strength to drop to zero and will usually cause significant reductions in shear strength as well as large increases in permeability. On a regional scale, discontinuities are largely responsible for the distinctive drainage patterns and major erosional features that are characteristic of faulted and jointed terrain.

In rocks, most slope failures are controlled by ever present discontinuities. Slope failures will propagate depending on the extent, pattern and types of discontinuities present in the rock mass. The orientation of these discontinuities in combination with the natural face of rock, shall bring about one or more failure mechanisms that may involve free fall, sliding or rotation of rock blocks. Therefore, discontinuities are critical for identification of potential failure mechanism in fractured rock masses. Important characteristics of discontinuities influencing the strength are orientation, spacing, size & shape of block, roughness, aperture, its in-fillings, wall strength, wall coating, and seepage through them.

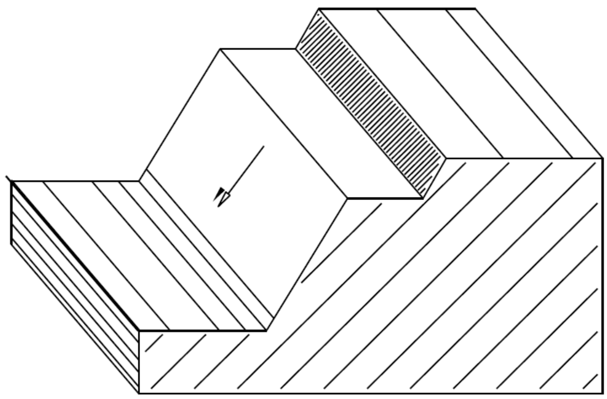
A rock mass may display one or more modes of failure depending on following factors:

a) Presence or absence of discontinuities
b) Orientation of discontinuities in relation to that of the natural or excavated face
c) Discontinuity spacing in one and three dimensions.
d) Shear strength of discontinuity walls
e) Persistence of discontinuities

Modes of Rock Failure at Portals:

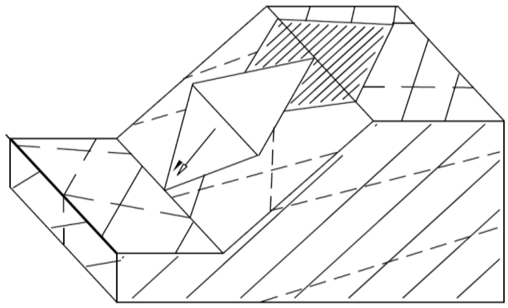
a) Plane Failure: It occurs when a geological discontinuity such as a bedding plane, strikes parallel to the slope face and dips into the excavation at an angle greater than the angle of friction. This is the one of the simplest modes of failure. For plane failure to occur in slopes there must be lateral release surfaces that will allow a block of finite size to slide out of the face. It occurs rarely in rock slopes.

PLANE FAILURE



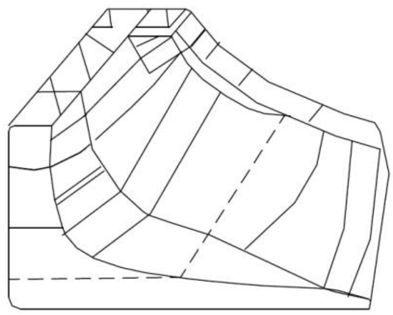
b) Wedge Failure: When two discontinuities strike obliquely across the slope face, the wedge of rock resting on these discontinuities will slide down the line of intersection, provided that the inclination of this line is significantly greater than angle friction. This is most dangerous mode of failure since no release surfaces are required. The calculation of factor in this case is more complicated than that for plane failure since the base areas of both failure planes as well as the normal forces on the planes must be calculated.

WEDGE FAILURE



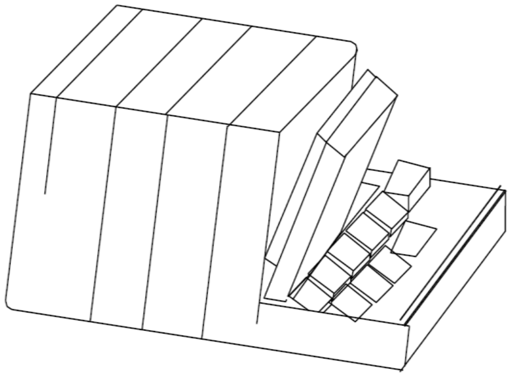
c) Circular Failure: It occurs when material is very weak or rock mass is heavily jointed or broken, the failure will be defined by a single discontinuity surface but will tend to follow a circular failure path. When the pattern of discontinuities is random circular failure modes are likely.

CIRCULAR FAILURE

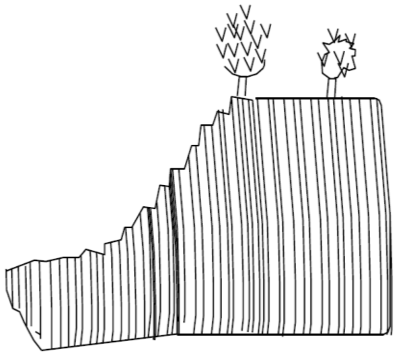


d) Toppling Failure: It occurs when vector representing the weight of block falls within the base, and inclination of plane is greater than angle of friction. Also, when rock block is tall and slender (height > width), the weight vector can fall outside the base and when this happens the block will topple i.e., it will rotate about its lowest contact edge. Toppling failure involves either one or a combination of flexural toppling and block toppling. Flexural toppling involves the overturning of rock layers like a series of cantilever beams. Block toppling involves the overturning of fracture-bounded blocks as rigid columns rather than having to fail in flexure.

BLOCK TOPPLING

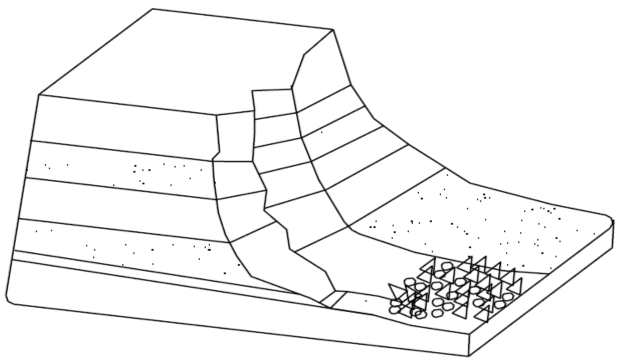


FLEXURAL TOPPLING



e) Rock Falls: Rock falls consist of free-falling blocks of different sizes which are detached from a steep rock face. The block movement includes bouncing, rolling, sliding and fragmentation. The problem in design of slopes from viewpoint of rock fall is the prediction of the paths and the trajectories of the unstable blocks which detach from the rock slope so that suitable protection measures are constructed well in advance.

ROCKFALLS



17 Design Methodology

It involves the design of following components:

a) Slopes- RDSO standard slopes has been provided as per the Geotech classification based on Geomapping, Geophysical, bore log details and site photographs.

b) Benching/Berms – RDSO standard for rocks minimum berm width of 4-6 m. It is advisable to provide berms in soil slopes at every 6 to 7 m height to break monotony of slopes. Width of first berm from formation may be kept as 5 m and that of subsequent berms as 4 m. In cuttings where soil-strata are in top portion and weathered/jointed rock in bottom portion, it is essential to provide 5 to 6 m berm at soil-rock interface.

c) erosion and slope protection work – providing retaining walls, steel wire net with u clamps and providing berms at adequate height interval.

d) side drains- Side drain width is generally standard 1.2 m wide on top, 0.6 m at bottom. The depth is min 0.3 m, with deeper drains as per longitudinal slope depending upon length of cutting. Sub-surface longitudinal drains may be required where blanket layer has been provided.

e) boundary drains- Rainfall, percolation or streams that flow outside the boundaries of the cutting have a grave potential of affecting the behaviour of the cutting in immediate or even distant future. Effective steps need to be taken to avert any eventuality of ingress of any such water that has not been catered for in the design. This job is to be accomplished by provision of boundary or peripheral drains. These drains are also called catch water drains.

18 Analysis Results

This is the preliminary stage design based on the data available from Geological mapping, and Geotechnical investigations. The models in RS2 have been analysed in piezometric conditions due to presence of water.

Table 13 FOS PORTAL SLOPE TUNNEL-8A

Portal Slope Type	**V:H**	**Support Recommended**	**Factor of Safety (Should be > 1.4 Static Case,>1.1 for Seismic Case**
Portal` P-1 /P-2 Tunnel-8A	3 V: 1 H	100 mm shotcrete + Wiremesh 150x150x6 mm along with spot bolting and array of relief holes and perforated drainage pipe	1.42 (Static) 1.17 (Seismic), Hence OK.

Hence following supports are provided in portal drawings.

1. The detailed analyses must be carried out during detailed design and further modifications can be made during construction stage based on the site conditions.
2. Excavated slope surface is assumed to be free draining and no water pressure shall be allowed to be built up behind the shotcrete sealing or in the ground close to the excavated face. For this, drainage measures such as drainage drillings and/or weep holes must be applied. It is critically important to implement and maintain proper drainage arrangement along the excavated slopes during the execution of the project.
3. The slope stability analyses are carried out for critical sections for portals. Any change in the portal location from what have been analysed would require rework of slope stability analysis.
4. Monitoring program including inclinometers and survey points shall be implemented during construction stage to detect any possible creeping or slope movements.

19 Portal cross- sections

Major rock cuts require detailed subsurface investigation to know the type & condition of rock strata before taking up the excavation. As and when excavation progresses, additional geological information helps in deciding rock slope, at various levels, by carrying out tests like compressive strength, petrographic examination of samples, soundness tests etc.

The blasting in rock strata plays a very significant role in slope stability. Uncontrolled blasting often results in shattering of rock mass, by means of opening of joints, developments of tension cracks, rough, uneven contours, overbreak's, overhangs etc. The results of blast shock wave, along various discontinuities can lead to loosening of the rock.

Rough guide for adopting the slopes or cuts in rock is given in table below. In adopting this table, caution must be exercised and such factors as the influence of dip in relation to the inclination of the slope face, the nature of joints etc. must be kept specially in mind. However, it would lead to safer and economical rock slopes if proper design methods are adopted to evaluate the stability of rock slopes as well.

Berms are provided to divide the long slopes into segments of short slopes which reduce pressure at the toe of the cuts, thus increasing the stability.

Slope cuts have been analysed by correlating their geotechnical properties with the range of permissible slope, and berms with bench heights as per RDSO standards. The cross section in tunnel approach both sides for 50 m is computed at the next page. Also, RDSO recommends 1:2 (H: V) in mixed Basalt rock conditions and same has been analysed with support system for calculation of factor of safety

Sl. No.	Rock Type	Range of permissible slope (H : V)
A.	**Sedimentary Rocks**	
1.	Massive sand stones and lime stones	0.25 : 1 to 0.50 : 1
2.	Jointed/Inter bedded/Layered sand stones, lime stone & shales	0.50 : 1 to 0.75 : 1
3.	Massive clay stone and silt stone	0.75 : 1 to 1 : 1
B.	**Igneous Rocks**	
1.	Massive Granites & Basalts	0.25 : 1
2.	Jointed Granite, Jointed Basalt	0.50 :1
C.	**Metamorphic Rocks**	
1.	Gneiss, Schist and Marble	0.25 : 1 to 0.50 : 1
2.	Slate	0.50 : 1 to 0.75 :1
D.	**Weathered Rocks (All types)**	1:1

Figure 7 Slope recommendation for slope stability

20 Conclusion

The rock support systems for Tunnel T-8A are so provided such that it satisfies the factor of safety criteria. Analysis result shows that at portal slopes are stable.

The geotechnical parameters considered for the analyses are sensitive to the site conditions and susceptible to further modifications during construction stage.

Excavated slope surface is assumed to be free draining and no water pressure shall be allowed to be built up behind the shotcrete sealing or in the ground close to the excavated face. For this, drainage measures such as drainage drillings and/or weep holes must be applied. It is critically important to implement and maintain proper drainage arrangement along the excavated slopes during the execution of the project.

The slope stability analyses are carried out for critical sections for portals. Any change in the portal location from what have been analysed would require rework of slope stability analysis.

Monitoring program including inclinometers and survey points shall be implemented during construction stage to detect any possible creeping or slope movements.

21 Construction Methodology for Tunnel T-8A

21.1 NATM Tunnel with drilling and blasting

NATM is recommended for this tunnel, as nearly all the tunnel sections are identified with hard rock masses, drilling and controlled blasting is proposed for tunnels with rock class III.

The construction methodology for “NATM Tunnel with drilling and blasting” includes following steps:

1. Excavating the ground profile at 3V:1H in layers of depth 2m at a time and providing support arrangement at the slope for slope stability as mentioned in report and detailed slope arrangement drawings before each next layer of excavation.
2. Ensuring provision of relief holes and perforated drainage pipe wrapped in geotextile membranes as Tunnel-8A is having water level at shallow depth.
3. Once the portal construction is complete, tunnel construction will commence.
4. False Portal shall be prepared as specified in drawings and portal support class to be followed for tunnel construction commencement.
5. Afterwards Line positioning.
6. drilling, loading, and controlled blasting.
7. dust removal by ventilation.
8. Installing primary supports such as shotcrete and spot bolts, wherever required. The support system may vary as per the site’s geological conditions and on recommendation of engineering geologist.
9. building the RCC lining as per support class by the secondary lining using moveable shuttering.

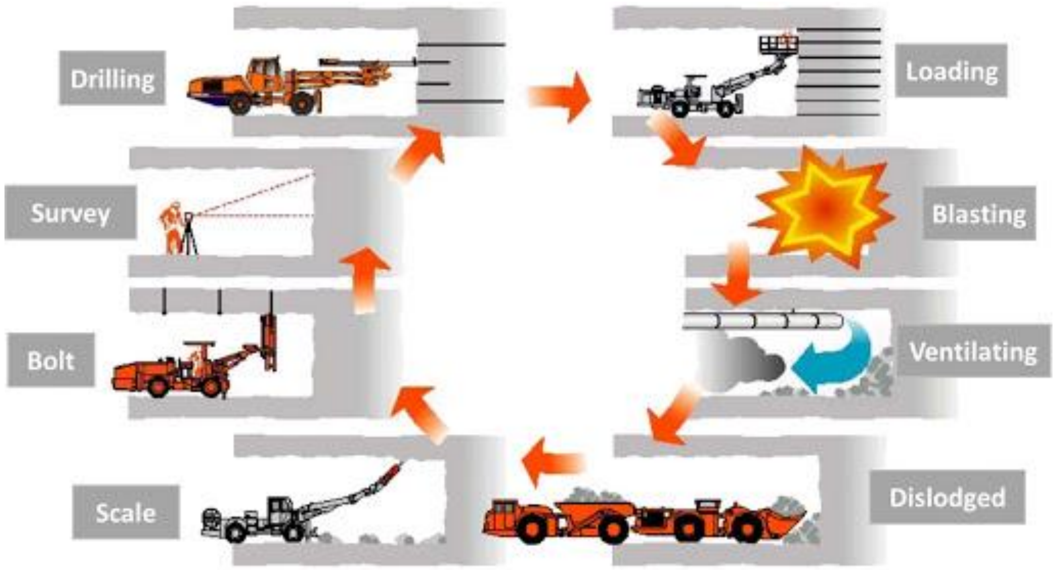


Figure 8 Construction Methodology for NATM Tunnels with mechanical excavation and drilling blasting.

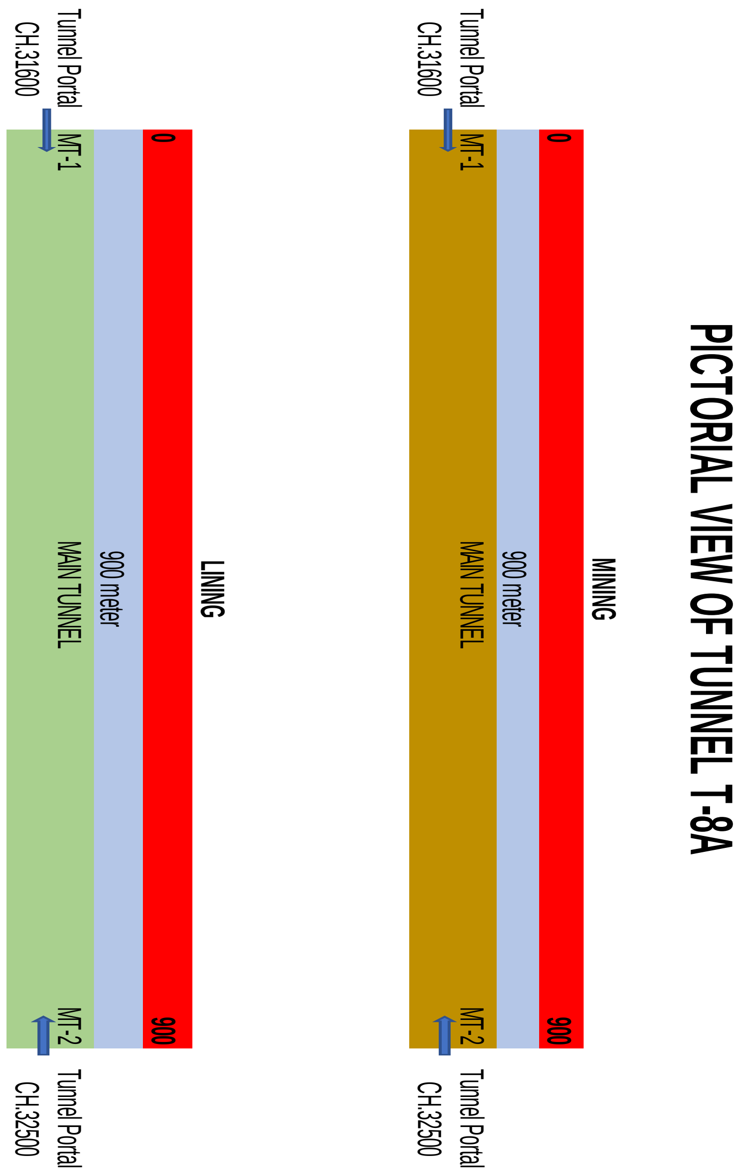


Figure 9 Construction Methodology planning from each phases as per Support class recommended

22 CONSTRUCTION PROGRAM CYCLE TIME ACCORDING TO VARIOUS ROCKCLASS

MAIN TUNNEL							
Sr. No	Activity Description	Units	Class 1	Class 2	Class 3	Class 4	Class 5
1	Survey & Marking	Minutes	60	60	60	60	60
2	Drilling	Minutes	133	93	97	75	0
3	Loading	Minutes	45	45	40	35	0
4	Blast	Minutes	5	5	5	5	0
5	Defuming	Minutes	30	30	30	30	30
6	Mucking	Minutes	191	130	135	110	115
7	Scaling	Minutes	30	30	30	30	0
8	Excavation of Face by Twin Cutter	Minutes	0	0	0	0	240
9	R/bolts & Forepoles	Minutes	0	33	44	53	181
10	Lattice girder	Minutes	0	0	0	60	90
11	W/mesh	Minutes	0	0	30	45	60
12	S.crete	Minutes	102	101	107	123	128
13	Down Time of Equipment	Minutes	90	90	90	90	90
14	**Total Time**	**Minutes**	**686**	**618**	**668**	**717**	**994**
15	**Time to complete 01 cycle**	**Hrs**	**11.4**	**10.3**	**11.1**	**11.9**	**16.6**
	Pull achieved	**Mtr**	**3.0**	**2.0**	**2.0**	**1.5**	**1.5**
Progress Achieved considering 20 hrs/Day		**Mtr**	**5.3**	**3.9**	**3.6**	**2.5**	**1.8**
Tunnel T-8A							
Length of Rock classification		**Mtr**	**0**	**150**	**580**	**170**	**0**
No. of Days required		**Days**	**0**	**39**	**161**	**68**	**0**
Progress per month (considering 28 days/month)		**Mtr**	**147**	**109**	**101**	**70**	**51**
Time taken		**Months**	**0.00**	**1.38**	**5.77**	**2.42**	**0.00**
Total Time Taken to complete the tunnel		**Months**	**9.56**				
Average Progress / month		**Mtr**	**94**				

23 CALCULATION OF CYCLE TIME

23.1 ROUND LENGTH FOR SUPPORT CLASS I TO V

Description	Unit	Class 1	Class 2	Class 3	Class 4	Class 5
Area of face	Sq. m	51.04	52.11	54.17	54.17	56.56
Area including swelling factor (1.5 to 1.63%)	Sq. m	76.56	78.165	81.255	88.2971	92.1928
Drilling of 1 m hole by boomer	Seconds	40	40	40	40	0
Removing drill rod from hole	Seconds	5	5	5	5	0
Placing rod for other hole	Seconds	5	5	5	5	0
Drilling of one complete hole	Seconds	134.4	89.6	89.6	67.2	0
Drilling length for one hole to achieve the desired pull	M	3.4	2.2	2.2	1.7	1.7
Total Time for one hole	Seconds	144.4	99.6	99.6	77.2	0
Total Time for one hole	**Minutes**	**2.41**	**1.66**	**1.66**	**1.29**	**0.00**
Excavation of Face by Twin Cutter/Road header	**Minutes**					**240**
Pull achieved	**M**	**3**	**2**	**2**	**1.5**	**1.5**
No. of holes to be drilled in face	**No's.**	**110**	**113**	**117**	**117**	**0**
Total time taken for Face drilling	**Minutes**	**265**	**187**	**194**	**151**	**0**
Drilling time by 2 booms	**Minutes**	**133**	**93**	**97**	**75**	**0**
Loading of face	**Minutes**	**45**	**45**	**40**	**35**	**0**
Blast	**Minutes**	**5**	**5**	**5**	**5**	**0**
Defuming	**Minutes**	**30**	**30**	**30**	**30**	**30**
Total volume of muck for the desired pull incl. swelling factor	**Sq. m**	**229.68**	**156.33**	**162.51**	**132.45**	**138.29**

23.2 MUCKING TIME CALCULATIONS

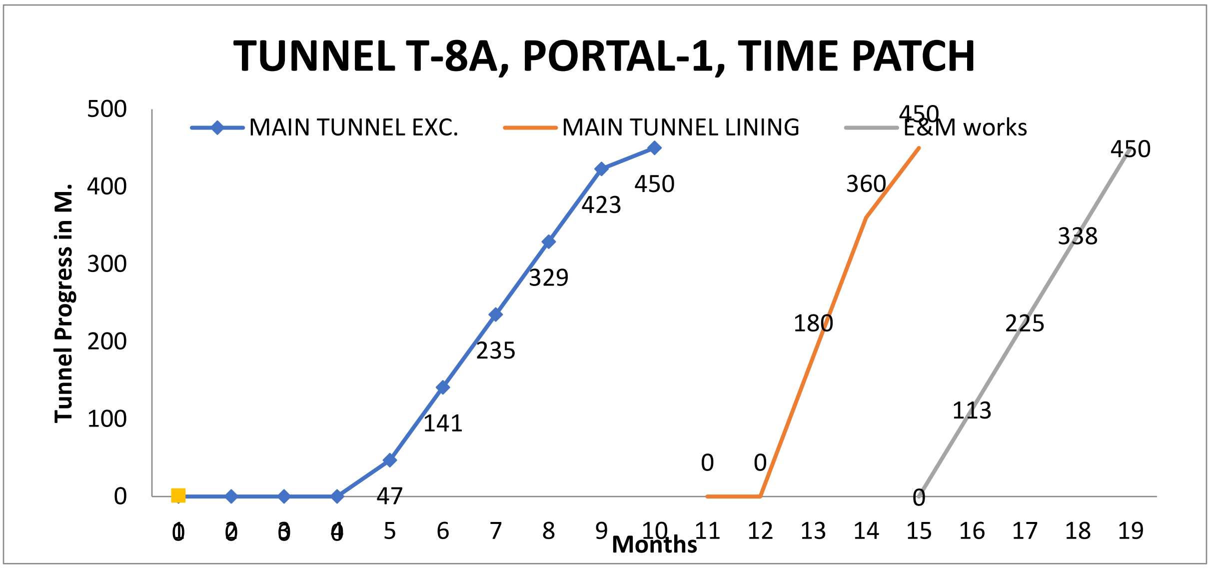
Activity Description	Units	Class 1	Class 2	Class 3	Class 4	Class 5
Assume Lead (Open + Tunnel)	Km	3	3	3	3	3
Excavator capacity	Cum/hr	45	45	45	45	45
Dumper capacity	cum/hr	10	10	10	10	10
Positioning of dumper	Min.	2	2	2	2	2
Loading of dumper	Min.	13	13	13	13	13
Travelling @ 15Km/hr	Min	8	8	8	8	8
Unloading of dumper	Min.	2	2	2	2	2
Return @20Km/hr	Min.	6	6	6	6	6
Misc. Time	Min.	2	2	2	2	2
Total Time	Min.	33	33	33	33	33
Qty. of muck hauled per unit per hour	Cum	18	18	18	18	18
Total vol. of muck	Cum	230	156	163	132	138
Assuming Dumpers	no's	4	4	4	4	4
Time taken to complete the muck	Hr	3.19	2.17	2.26	1.84	1.92

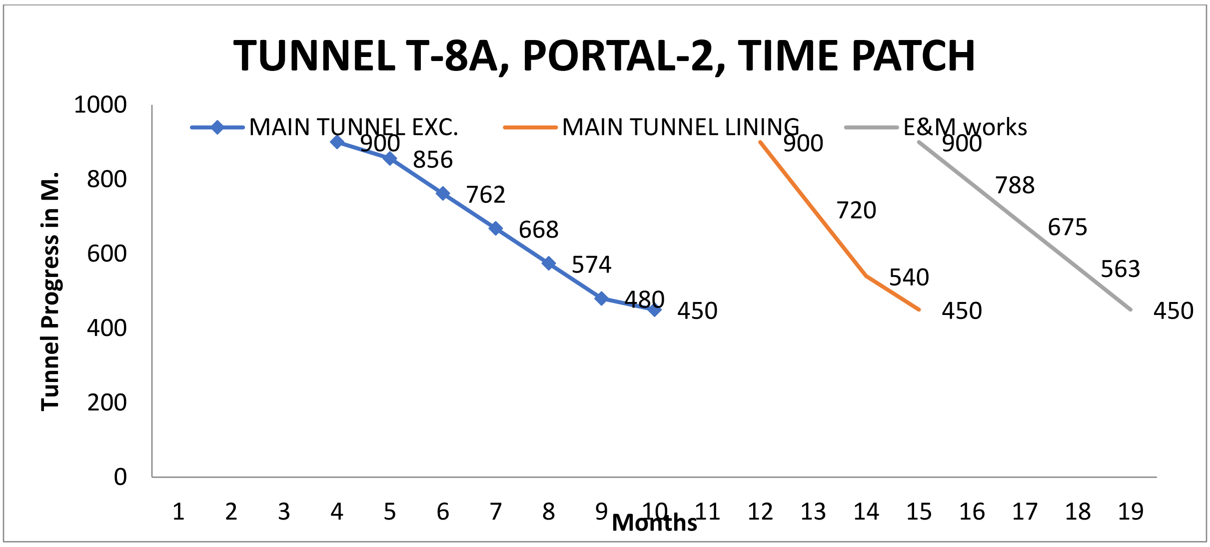
23.3 ROCK SUPPORT TIME CALCULATIONS

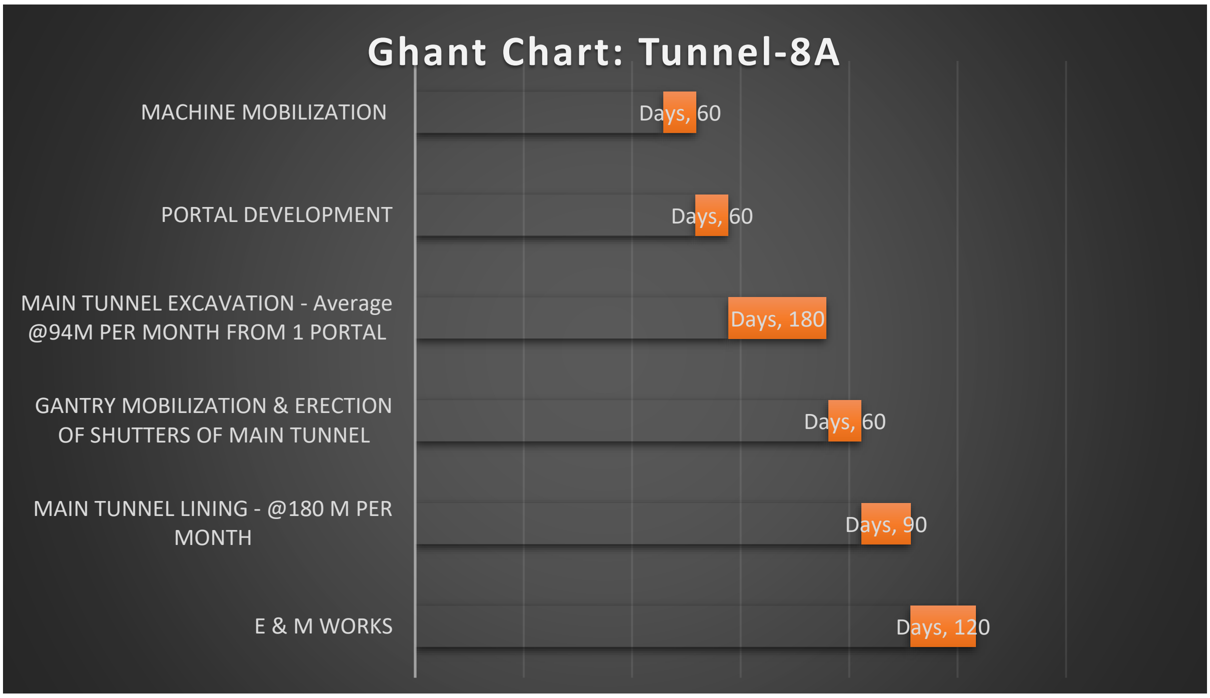
Description	Unit	Class 1	Class 2	Class 3	Class 4	Class 5	Column1
		R/Bolts	**R/Bolts**	**R/Bolts**	**R/Bolts**	**R/Bolts**	**Forepoles**
Drilling of 1 m hole by boomer	Seconds		40	40	40	40	40
Removing drill rod from hole	Seconds		5	5	5	5	5
Placing rod for other hole	Seconds		5	5	5	5	5
Drilling of 3 m hole	Seconds		120				
Drilling of 4 m hole	Seconds			160	160		160
Drilling of 6 m hole	Seconds					240	
Total Time for one hole	Seconds		130	170	170	250	170
Inserting R.bolts incl. Resin capsules	Seconds		120	120	120	150	120
Total Time for one hole	**Min.**		**4.17**	**4.83**	**4.83**	**6.67**	**4.83**
No. of R/bolts	**No's**		**8**	**9**	**11**	**9**	**25**
Total Time	**Min.**	**0**	**33**	**44**	**53**	**60**	**121**

Other Supports in Tunnel T-8A							
Activity Description	**Units**	**Class 1**	**Class 2**	**Class 3**	**Class 4**	**Class 5**	**Mechanical Excavation**
Lattice Girder	Minutes	0	0	0	60	90	60
Wiremesh	Minutes	0	0	30	45	60	60
Shotcrete	Minutes						
Surface area for shotcrete	M^2	20.2	20.2	20.3	20.43	21.07	21.07
Thickness of shotcrete	mm	50	50	100	200	200	200
Qty. of shotcrete /mtr	Cum	1.0	1.0	2.0	4.1	4.2	4.2
Qty. of shotcrete for one round length	Cum	3.03	2.02	4.06	6.13	6.32	3.16
Shotcrete M/c set-up	Minutes	30	30	30	30	30	30
T.M arriving time from B/Plant (Parallel activity)	Minutes	0	0	0	0	0	0
T.M positioning	Minutes	5	5	5	10	10	5
Shotcrete Spraying assume 30m3/hr	Minutes	16.515	16.010	22.030	33.065	38.161	16.580
Shotcrete Machine removing	Minutes	20	20	20	20	20	20
Total Time taken	**Minutes**	**72**	**71**	**77**	**93**	**98**	**72**
Breakdown/ Maintenance	**Minutes**	**30**	**30**	**30**	**30**	**30**	**30**
Final Time taken	**Minutes**	**102**	**101**	**107**	**123**	**128**	**102**

23.4 CONSTRUCTION PRORGRAM TUNNEL 1A







The Total time for construction of Tunnel-8A is calculated to be 19 months if the scheme recommended is followed judiciously.

24 Objective of Ventilation in tunnels

The objective of the ventilation report is to access the tunnel for requirement of artificial ventilation. The report discusses the allowable pollutants (dust in the tunnel and polluting gases) in various case scenarios such as while tunnel is empty, when a train has travelled with a diesel locomotive and in case of fire. For each of these cases, it is assessed that whether artificial ventilation is required or not. Further, an assessment is made on the required jet fan specifications for ventilation purposes. As the tunnel length is 900 m, ventilation system is not required. **Annexure-3** (Not Applicable in this tunnel)

25 Item Rate Cost Estimate of Tunnel-8A (Civil Works, E&M, Ventilation)

The item rate cost estimate is based on 3 LAR's of recently awarded tunnel contracts based on Item Rate Model whose DPR and Tender preparation is done By RITES Limited. Western Railways is requested to review the cost estimate and provide LAR of the tunnel contracts in their section to finalize the cost estimate, as per Railway guidelines, first priority in referring to LAR for cost estimate shall be given to the similar section in nearby territory in which contract is to be awarded. **Refer Annexure-4** for details.

26 Recommendation

While open cutting is often considered a practical approach in regions with minimal overburden and flat terrain due to lower initial construction costs and simplicity, the decision to propose a tunnel in this case is based on a thorough assessment of site-specific technical and operational considerations:

1. **Future Maintenance Optimization**

Open cuts, particularly in areas with significant precipitation, demand extensive maintenance, including erosion control, slope stabilization, and removal of debris. In contrast, a tunnel offers a more durable solution by eliminating the need for frequent interventions to address weather-induced wear or slope instability. This significantly reduces the lifecycle costs of the infrastructure.

2. **Hydrological Challenges and Dewatering Efficiency**

   During the monsoon season, open cuts are prone to substantial water ingress due to surface runoff and groundwater exposure. Managing this requires extensive dewatering systems, which are operationally intensive and add to maintenance complexity. A tunnel, being an enclosed structure, minimizes water accumulation, relying primarily on controlled drainage systems, thereby offering a more predictable and manageable solution.

3. **Geotechnical Stability and Safety**

   The geological profile of the region suggests the likelihood of loose or fragmented rock strata, particularly in areas with weathered or fractured zones. Open cuts in such conditions are susceptible to rockfalls and slope failures, posing safety risks during construction and operation. A tunnel mitigates these risks by stabilizing the surrounding rock through engineered lining systems, ensuring a higher degree of safety and structural reliability.

4. **Environmental and Aesthetic Considerations**

   Tunnels typically have a reduced environmental footprint compared to extensive cuts, which may involve significant deforestation, land acquisition, and ecological disruption. Furthermore, the visual and acoustic impacts of rail infrastructure are minimized, aligning with modern sustainability and community acceptance goals.

By prioritizing a tunnel design, the railway aims to address the challenges associated with open cuts while ensuring a robust, safe, and sustainable infrastructure solution tailored to the region's geological and hydrological conditions. This approach also aligns with long-term operational efficiency and environmental stewardship.