A REPORT ON
STRUCTURAL ANALYSIS, GEOTECHNICAL
ASSESSMENT OF COMPOUND WALL FAILURE
PHORA COMPOUND, US EMBASSY
KANTIPATH, KATHMANDU, NEPAL
AUGUST 2021

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EXECUTIVE SUMMARY

This report is prepared by Earthquake Safety Solutions (ESS) as a part of report on structural and geotechnical assessment of unreinforced masonry wall failure on the northern boundary of Phora Durbar. Around 15-20 m length of the boundary wall is toppled on the side of Annapurna hotel’s construction site. The compound wall is observed to be unreinforced with no presence of horizontal bands and slenderness of the wall which caused the wall to be toppled on the Annapurna hotel side due to lateral movement of the soil on the premise caused by unexpended supports from the sheet piles.

Based upon the information provided from US Embassy, Kathmandu, ESS has validated the member sizes its availability for the super structure in different options. Whereas for sub structure ESS has recommended the different options based upon the engineering judgement and calculations. Fundamentally, the selection of the options will depend upon the nature of construction, in terms of duration. The provided data were analyzed as the desk study and site visit was carried out on 30th of the July as a field walk through visual observations.

The provided superstructure fence is able to withstand the wind and vertical load. The substructure has been proposed of its type and it has been recommended to adopt option .

This report solely restricts itself to the technical aspects of the boundary wall and does not comment on other aspect of the structure.
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CHAPTER 1. INTRODUCTION

1.1 General

This report on structural and geotechnical assessment of unreinforced masonry wall failure on the northern boundary of Phora Durbar is prepared by Earthquake Safety Solutions (ESS). This report presents the detail structural and geotechnical outcomes from the field assessment carried out on 30th July 2021. The compound wall is observed to be unreinforced with no presence of horizontal bands and slenderness of the wall which caused the wall to be toppled on the Annapurna hotel side due to lateral movement of the soil on the premise caused by unexpended supports from the sheet piles. ESS has revisited on September 21-2021 to verify the location of the trees, poles and existing structures along with the verification of the SOE works done by the Annapurna’s side.

ESS has evaluated the approach presented by the US Embassy for the super structure and presented the option for the sub structure part. Based upon the temporary nature of the boundary walls the path selection is chosen in such a way that there will be enough space for the construction and the trees, electric poles are also avoided. The temporary design approach will be for the short term to be built 500 mm to 2500 mm offset from the collapsed wall whereas when the construction work on the Annapurna side is completed permanent wall will be constructed on the property line of the Phora Durbar. The temporary fence is connected with the RCC pillar on the North side and its separated from the old (to be demolished on future) boundary wall.

Table 1-1 General Information

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Unreinforced Masonry wall with Buttresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height of the Wall</td>
<td>2.5 m with 1 m MS fence on top</td>
</tr>
<tr>
<td>Width of the Wall</td>
<td>350 mm at Mid, 550 mm at Buttress</td>
</tr>
<tr>
<td>Slenderness Ratio</td>
<td>6.85</td>
</tr>
<tr>
<td>Spacing of Buttresses</td>
<td>1.2 m c/c</td>
</tr>
<tr>
<td>Type of foundation</td>
<td>500x500x600 Block</td>
</tr>
<tr>
<td>Terrain</td>
<td>Flat terrain with lake deposit inside Kathmandu Valley</td>
</tr>
<tr>
<td>Name of Existing Drain</td>
<td>Water Logged during monsoon (due to construction activities at Annapurna Hotel Premises)</td>
</tr>
<tr>
<td>Soil Type (on surface Observation)</td>
<td>Light to Dark grey loose to medium dense clayey silt</td>
</tr>
<tr>
<td>Geographical coordinates</td>
<td>27°42'42.81&quot;N, 85°19'0.84&quot;E</td>
</tr>
</tbody>
</table>
1.2 Location of the site

Phora Durbar Site lies in the heart of Kathmandu Valley just in the Eastern boundary of Narayanhiti Durbar road and Northern boundary of Kantipath Marg. The compound wall failure site can be accessed through the premises of Annapurna Hotel. The physical facilities such as Electricity, Water Supply, Communication etc are easily available at Site. There is a building nearby the Compound wall so the disturbance during construction works need to be identified and tried to minimize.

Referring to the available Engineering and Environmental map of the Kathmandu valley, Scale (1:50,000), published by Department of Mines and Geology, Nepal in 1998. The following geological information about the site can be studied. In general, the Kathmandu Basin lies on the Kathmandu Nappe (Hagen, 1952) along the southern slopes of the Himalaya. A lake occupied a large part of the basin from Pliocene to Pleistocene (Yoshida and Igarashi, 1984). The basin is currently filled with a very thick (500–600 m) sequence of fluvio-lacustrine sediments (Moribayashi and Maruo, 1980) and is bounded to the south by a tectonic ridge developed above the Main Boundary Thrust (MBT). On the northern part of the valley, sediments are poorly sorted, thin to medium-bedded highly micaceous coarse sands, gravel, and silts interlayered with clays. In the south, they consist of a thick sequence of dark grey to black highly plastic clay and silts, usually overlain and underlain by coarse sediments. The black plastic clay (locally called Kalimati or black cotton) is rich in organic matter. The age of this clay is placed in the Pliocene to Pleistocene time according to Yoshida and Igarashi (1984). According to the same study, the maximum thickness of the Black Clay is approximately 300 m, and is greatest along the central part of the valley starting from Satungal towards Lalitpur and Bhaktapur. As per the Geological map of Kathmandu valley, the project site lies in Gokarna formation. Gokarna Formation consists of light to brownish grey; fine laminated and poorly graded silty sand intercalations of clay of variable thickness as well as in upper part. Locally prone to erosion and flooding. Moderate to high ground water potential.
1.3 Objective

The objective of the project is to perform the analysis and recommend the foundation design of the building as per the codal provisions or based upon the engineering judgement.

The specific objectives include the following:

i. To prepare the calculations if any

ii. To prepare the detail structural, geotechnical report and drawings.

iii. To prepare the detail estimate of the metal fence, concrete work.

1.4 Scope of Work

The precise scope of the work is as follows:

1. Collect the available documents from US Embassy, understanding the requirement for metal fence.
2. Schedule a visit to the site structural and geotechnical team to inspect the deflection of the wall, its cause.
3. Inspect the sheet piling and ISNB piling done from the hotel Annapurna side.
4. Inspect the location for the new temporary fence site.
5. Connection to the existing portion of the wall.
6. Prepare the estimate for the amount of steel used in proposed option.
1.5 Limitations

This report is prepared by Earthquake Safety Solutions (ESS). Since the detailed investigation was not carried out, some uncertainties remain regarding the actual consequences.
2.1 Failure of Compound Wall

During the site visit on July 30th 2021 and 16th September 2021 it is observed that the substructure construction work (Piling works) at the adjacent boundary of Phora Durbar compound wall is underway. We have noticed that the construction works within the adjacent boundary has been for long term about 2 years. Generally, the temporary sheet piling works during substructure construction has been designed for 6-9 months, however in this particular site, the construction work of sub-structure has been extended for about 2 year due to different causes and the bulging of sheet pile has been observed due to improper technology of sheet pile construction.

The self weight of compound wall, weak soil strata at the foundation wall has added shearing forces and collapse of top portion of sheet pile unsupported by circular pipes and ultimately compound wall.

2.2 Mitigation Measures

ESS has planned for short-term protection and construction of compound wall until the adjacent Annapurna Hotel building construction works complete. The short-term plan is to construct the temporary boundary wall with steel pipe support of 6 m inside Phora Durbar Premises. Hence, the proper foundation needs to be constructed before constructing the steel pipe support. It has been observed from the previous Annapurna Hotel premises soil investigation report that the soil strata is very poor and water table is high(3.45 m). So, we would suggest for the construction of deep foundation even for temporary boundary. The reason behind choosing the deep foundation is that it protects the further bulging of sheet pile, Stabilize the washout out of soil with high water table and it will act independent structure component to bear the load of proposed new compound wall.
2.2.1 **Option-IV**

As an option to less intervent and disturb the natural soil conditions, post driven steel pipes are hammered into the ground with help of air compressor or mechanical equipment. Holes shall be pre-drilled with 100mm diameter auger and shall be pushed into limit vibrations.
Based on calculations from section 2.6 of the report depth of 7 m is proposed with ISNB 125 5.4 mm thickness.

2.3 Material Strength

2.3.1 Partial Safety Factor for Materials

The design strength of concrete or reinforcing steel is obtained by dividing the characteristic strength by the appropriate partial safety factor. For the ultimate states, the code specified partial safety factors are 1.5 and 1.15 for concrete and steel respectively. The unit weights of the materials for the proposed buildings are assumed as follows as per IS 875 part I (dead load) code. The grade of concrete, steel and unit weight of materials is specified in the Error! Reference source not found., Table 2-1 and Table 2-2 respectively.

2.3.2 Structural Steel

Table 2-1: Steel Grade

<table>
<thead>
<tr>
<th>Member</th>
<th>Steel Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel</td>
<td>Fy250</td>
</tr>
</tbody>
</table>
2.4 Loading Criteria

The minimum dead and live loads are calculated as per IS 875(part-1)-1985 and IS 875 (part-
2) -1987. The self-weight of the steel members are taken as per IS800-2007 steel code. Wind
load is taken from ASCE 7-16 provided from the OBO standard.

2.4.1 Gravity Loading

Gravity loading on the structure comprises of the self-weight of the member, weight of the
floor finishes, partition walls.

Table 2-2: Unit Weight of Material

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Unit Weight (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of Structural Steel</td>
<td>78.5</td>
</tr>
<tr>
<td>Soil</td>
<td>26 (from soil report)</td>
</tr>
</tbody>
</table>

2.4.1.1 Dead Loads

The dead loads in the building are assumed as per IS 875-part I (Dead Loads) code and IS 800-
2007 Code. The detailed calculations are shown under design calculations section.

2.4.2 Seismic Load

For fence seismic load is less critical since its of steel instead of masonry, so calculations only
for wind loads are done.

2.4.3 Wind Load

| Wind load Calculations as per OBO Standard- ASCE 7-16 |
|-----------------------------------------------|--------|-------|-------|-------|
| SN   | Parameters          | Symbols | Value | Unit | Remarks       |
| 1    | Basic wind Speed    | V$_h$   | 57.00 | m/sec| OBO Standard  |
| 2    | Exposure Type       | B       | B     |      |               |
| 3    | Exposure Coefficient| K$_z$   | 0.70  |      | Section 27.3.1|
| 4    | Importance Factor   | I       | 1.00  |      |               |
| 5    | Topographical Factor| K$_t$   | 1.00  |      | Section 26.8  |
| 6    | Directionality Factor| K$_d$  | 0.85  |      | Section 26.7  |
| 7    | Velocity Wind Pressure| q$_v$ | 1185.02 | N/m$^2$ | 0.613 V2 I Kz Kzt Kd |
| 8    | Guest Effect Factor | G       | 0.85  |      | Section 26.9  |
| 9    | Wall Pressure Coefficient| C$_p$ | 0.80  |      |               |
| 10   | Design Wind Pressure| q$_d$ GC$_p$ | 805.82 | N/m$^2$ |               |
2.4.4 **Partial Safety Factor for Load**

Following load combinations have been adopted with factor of safety of 1.5:

![Figure 2-6 Assumed Load distribution for the vertical and Horizontal Member](image-url)
2.5 Design Calculation for Fence

![Diagram of Chain Link Fence](image)

*Figure 2-7 Chain Link Fence*

### 2.5.1 Load Calculations

**VERTICAL POST DESIGN:**

#### Design Of Compression Members

<table>
<thead>
<tr>
<th>SN</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Quantity</th>
<th>Units</th>
<th>Reference &amp; notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Factor Load in</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>Member</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max Factored Axial Force</td>
<td>( P_u )</td>
<td>5.50</td>
<td>kN</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max factored BM in Zdir</td>
<td>( M_z )</td>
<td>3.46</td>
<td>kNm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max factored BM in Ydir</td>
<td>( M_y )</td>
<td>0.00</td>
<td>kNm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Column Height</td>
<td>( L )</td>
<td>2752.34</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>Member Geometry</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Grade of Steel</td>
<td>( f_y )</td>
<td>250.00</td>
<td>MPa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Depth of Column</td>
<td>( h )</td>
<td>100.00</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Breadth of Column</td>
<td>( b_f )</td>
<td>100.00</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Effective Area of Column</td>
<td>( A_e )</td>
<td>1095.00</td>
<td>mm²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thickness of Flange</td>
<td>( t_f )</td>
<td>5.40</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thickness of Web</td>
<td>( t_w )</td>
<td>5.40</td>
<td>mm</td>
<td></td>
</tr>
</tbody>
</table>
**Half Width of Flange**

\[ B = \frac{(bf - tw)}{2} = 47.30 \text{ mm} \]

**Axial Capacity of Column**

(Consider one end fix at foundation and another end pin at top)

a) **Slenderness Ratio**

\[ K = \frac{0.80}{\text{Table 11}} \]

\[ r = \frac{38.60}{\text{mm}} \]

\[ KL/r = 57.0 \]

**Check** = OK

b) **Buckling Class**

\[ h/bf = \frac{1.00}{\text{Table 10}} \]

Buckling Class of Selected beam in ZZ minor axis is "c", and in YY major axis is "c".

c) **Axial force capacity check for Buckling class "c" in ZZ minor axis direction.**

\[ F_{cd} = \frac{182.00}{\text{N/mm}^2} \]

\[ P_{da} = 199.3 \text{ kN} \]

**Check** = Since $P_{da} > P_u$, SAFE in ZZ Dir, so no Buckling will occur.

d) **Axial force check for Buckling class "c" in YY major axis direction.**

\[ KL/r yy = 57.0 \]

For $KL/r$ of 57.04 & $F_y = 250$

\[ F_{cd} = 182.00 \text{ N/mm}^2 \]

\[ P_{da} = 199.3 \text{ kN} \]

**Check** = Since $P_{da} > P_u$, SAFE in YY Dir, so no Buckling will occur.

**4.0 Moment Capacity Check**
Moment Capacity Check on ZZ

a) Major axis Dir
Section Classification from table 2 of IS-800:2007

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Quantity</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b/t_f )</td>
<td>18.52</td>
<td></td>
</tr>
<tr>
<td>( d/t_w )</td>
<td>18.52</td>
<td></td>
</tr>
</tbody>
</table>

For Plastic Section, \( \beta_p = 1 \), Semi compact section \( B_p = \frac{Z_e}{Z_p} \)

- \( B_p = 1.00 \)
- \( Z_e \text{ value} = 40400.00 \text{ mm}^3 \)
- \( m_0 = 1.10 \)
- \( M_{dzz} = 9.18 \text{ kNm} \)

Since \( M_{dzz} > M_z \), So SAFE.

Moment Capacity Check on YY

b) Major axis Dir

- \( B_p = 1.00 \)
- \( Z_e \text{ value} = 40400.00 \text{ mm}^3 \)
- \( M_{dyy} = 0.00 \text{ kNm} \)

Since \( M_{dzz} > M_z \), So SAFE.

5.0 (Utilization Ratio Check)

\( \frac{P_u}{P_d} + \frac{M_x}{M_{dxx}} + \frac{M_y}{M_{dyy}} = 0.40 \)

Check = Safe

Only 4% so it's OK

The vertical Fence post satisfies the demand in both of the fence

### DESIGN OF HORIZONTAL MEMBERS:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Quantity</th>
<th>Units</th>
<th>Reference &amp; notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored Load</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clear Span</td>
<td>( L )</td>
<td>2.90</td>
<td>m</td>
<td>From Analysis</td>
</tr>
<tr>
<td>Major Bending Moment</td>
<td>( M_u )</td>
<td>9.57</td>
<td>kNm</td>
<td></td>
</tr>
<tr>
<td>Minor Shear Force</td>
<td>( V_u )</td>
<td>0.00</td>
<td>kN</td>
<td></td>
</tr>
<tr>
<td>Major Shear Force</td>
<td>( V_u )</td>
<td>17.02</td>
<td>kN</td>
<td></td>
</tr>
<tr>
<td>Grade of Steel</td>
<td>( F_y )</td>
<td>250.00</td>
<td>Mpa</td>
<td></td>
</tr>
</tbody>
</table>

| Member Geometry | | | | |
| Section Properties | | = | 100x100x5.4 |
| Weight per Meter | \( W/m \) | 14.41 | Kg/m |
| Sectional Area | \( a \) | 1095.00 | cm\(^2\) |
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Depth of Section \( h = 100.00 \) mm
Width of flange \( b = 100.00 \) mm
Thickness of Flange \( t_f = 5.40 \) mm
Thickness of Web \( t_w = 4.00 \) mm
Moment of Inertia \( I_{zz} = 274.54 \) cm\(^4\)
\( I_{yy} = 274.54 \) cm\(^4\)
Raddi of Gyration \( r = 3.86 \) cm
Modulli of Section \( = 40.04 \) cm\(^3\)

**Slenderness Ratio Check**

Effective Length \( KL = 2610.00 \) mm
Minimum Radius of Gyration \( r_{\text{min}} = 38.60 \) mm
Slenderness Ratio \( KL/r_{\text{min}} = 67.62 \)
Check = Ok

**Buckling Capacity Check**

*Section Classification check for checking flange/web buckle and cripple possibility (Compactness Check)*

*If \( b/t_f \) and \( d/t_w \) not ratio exceeds Class-I, then buckling and cripling will not happen.*

\( \varepsilon \) (Eslon) = \((250/fy)^{0.5}\)

Flange Buckling Class Check
Breadth to Depth Ratio \( B/t_f = 8.17 \)
Class of Section = Class I
Check = Ok

Web Section Class
Depth of Web to Thickness Ratio \( d/t_w = 22.30 \)
Web section class = Class I
Check = Ok

**Check for Shear Capacity**

Design Shear Capacity \( V_d = 3.57 \) kN
Effective Shear Force \( V_d(\text{eff}) = 2.143 \) kN
Check = Safe

**Check for Moment Capacity**

Design Mom. Capcity (\( M_d \)) = 9.828 kNm
Check = Safe

**Check for Deflection**

Actual Deflection Delta (\( \Delta \)) = 12.48 mm
Allowable deflection Delta (allow) = 14.50 mm
Check = Safe

**Buckling Class Checks**

**Flange Buckling Check**

Flange cantilever beam length \( l_c = 0.05 \) m
Total Load \( W = 20.00 \) kN/m
Moment
\[ M = 0.022373 \text{ kN-m} \]
BM = \( Wl^2/2 \)
Flange Thickness
= Cant. Beam
\[ Y = 0.005400 \text{ m} \]
Depth
\[ I = 0.000006 \text{ m}^4 \]
(Deflection formula for cantilever beam)

Deflection Actual
\[ WL^4/8EI = 0.10358 \text{ mm} \]
Deflection Allowable
\[ L/150 = 0.3153 \text{ mm} \]
Check = Safe
Since Actual deflection is < Allowable Deflection, No required stiffener between flanges.

**Check for Web Buckling**
Depth of Web
\[ h = 500.00 \text{ mm} \]
Thickness of Web
\[ t_w = 9.90 \text{ mm} \]
Our rolled section ISWB 500
\[ h/b_t = 2.00 > 1.2 \]
Buckling Class
Buckling class for Z-Z axis
= a
Buckling class for Y-Y axis
= b
Since we are calculating the buckling of web, it falls under Y-Y axis, thus buckling class-b to be used in Table 9-B
Slenderness Ratio
\[ K_l/r = 167.34 \]
Buckling Stress, from Table-9B
\[ F_{cd} = 51.50 \]
Web Buckling Resistance
capacity of Section ISWB500
\[ F_{cdw} = 254.93 \text{ KN} \]
Check
\[ F_{cdw} = h \times t_w \times F_{cd} \]
Since \( Vu \) (Actual Shear) < \( F_{cdw} \) (capacity to take buckling shear), SAFE in web buckling. So no stiffener plate required.

**Check for Web Cripling**
Web Cripling Resistance of
Section
\[ F_w = 254.93 \text{ KN} \]
Check
\[ F_w = (h/2) \times t_w \times F_{cd} \]
Check = Safe
2.6 Post Driven Pile Analysis

From the Soil report it is found that silt to medium sand is observed up to the 14 m depth. So, Calculations are carried out for Silt-sandy Soils.

**Inputs:**
- Angle of Friction, $\phi$ = \textcolor{orange}{29} Degree (Soil report)
- Coefficient of active earth pressure, $K_a$ = \textcolor{orange}{0.347} Unitless
- Coefficient of passive earth pressure, $K_p$ = \textcolor{orange}{2.882} Unitless
- Density of saturated soil, $\gamma_{sat}$ = \textcolor{orange}{26} KN/m^3
- Density of the water, $\gamma_w$ = \textcolor{orange}{9.8} KN/m^3
- $\gamma' = \gamma_{sat} - \gamma_w$ = \textcolor{orange}{16.2} KN/m^3

From Figure Above:

**Inputs:**
- $P_1$ = $\gamma' L K_a$ = \textcolor{orange}{31.12} KN/m^2

**CANTILEVER EMBEDDED POST DESIGN**

From the Soil report it is found that silt to medium sand is observed up to the 14 m depth. So, Calculations are carried out for Silt-sandy Soils.
\[ p_2 = \left( L_1 + \gamma L_2 \right) k_a \]

\[ P2 = 31.12 \text{ KN/m}^2 \]

\[ p_3 = \frac{\gamma L_4}{k_p - k_a} \]

\[ P3 = 165.83 \text{ KN/m}^2 \]

\[ p_4 = p_5 + \gamma L_4 \left( k_p - k_a \right) \]

\[ P4 = 455.47 \text{ KN/m}^2 \]

\[ p_5 = \left( L_1 + \gamma L_2 \right) k_p + \gamma L_3 \left( k_p - k_a \right) \]

\[ P5 = 289.64 \text{ KN/m}^2 \]

Length of pile above water table, \( L_1 \)

\[ = 3.45 \text{ m} \]  
From report

Length of pile between WT and Dredge line, \( L_2 \)

\[ = 0 \text{ m} \]  
Assuming

Length of pile between point D and E, \( L_3 \)

\[ = 0.76 \text{ m} \]  
Goal Seek for \( L_4 \) to 0

Length of pile between point E and B, \( L_4 \)

\[ = 4.04 \text{ m} \]  
Coefficients

Length of pile between point F and B, \( L_5 \)

\[ = 0.87 \text{ m} \]  
Coefficients

\[ P = \text{Area of ACDE} = \frac{1}{2} p_2 \frac{L_2}{\left( k_p - k_a \right)} \]

\[ = 65.48 \text{ KN} \]  
Coefficients

\[ A_1 = \frac{1}{2} p_1 L_1 + \frac{1}{2} (p_2 - p_1) L_2 + \frac{1}{2} p_2 L_3 \]

\[ = 1.66 \text{ m} \]  
Coefficients

\[ A_2 = \frac{8P}{\left( k_p - k_a \right)} \]

\[ = 7.05 \]  
Coefficients

\[ A_3 = \frac{\left( 2z' (k_p - k_a) + 2p_3 \right)}{\left( k_p - k_a \right)} \]

\[ = 99.14 \]  
Coefficients

\[ A_4 = \frac{\left( 6p_2 k_p + 4P \right)}{\left( k_p - k_a \right)} \]

\[ = 121.85 \]  
Coefficients

Approximate embedment depth of pile, \( D \)

\[ = 4.59 \text{ m} \]  
Capacity of Single

\[ z' = \sqrt{\frac{2P}{\left( k_p - k_a \right)}} \]

\[ = 1.79 \text{ m} \]  
17.10 KN-m

Maximum Moment, \( M_{\text{max}} \)

\[ = 186.34 \text{ KN-m} \]  
Recommended Sheet Pile:

Allowable stress for sheet Pile

\[ = 250 \text{ N/mm}^2 \]  
ISNB 150 mm bore is taken

Surcharge due to Vehicular Movement

\[ = 0 \text{ KN/m} \]  
Provided Sectional modulus of sheet pile

Load due to Surcharge

\[ = 0.00 \text{ KN/m} \]  
\[ = 796.4 \text{ cm}^3 \]  
SAFE

Bending Moment due to Surcharge

\[ = 0.00 \text{ KN-m} \]  
Actual Depth of Embedment(\( D \))

Total Moment(\( M_{\text{total}} \))

\[ = 186.34 \text{ KN-m} \]  
\[ = 745.37 \text{ cm}^3 \]  
Recommended Sheet Pile:

S(Minm. required sectional modulus of pile)

\[ = 4.80 \text{ m} \]  
\[ = 796.4 \text{ cm}^3 \]  
SAFE
Sectional thickness, \( t \) = 6.0 mm
Effective width of the sheet pile, \( W \) = 150.0 mm
Effective height of the sheet pile, \( h \) = 300.0 mm
Embedment of the post driven pile = 8.04 m
Capacity of single Pile = 17.10 KN-m
Maximum Demand on Single Pile = 7.70 KN-m
Demand to Capacity Ratio = 0.45 KN < 1 O.K
Depth of Embedment = 4.04 m from Annapurna side
Total Depth of Embedment = 6.54 m from Phora Side

2.7 Assumptions

In the analysis following assumptions are made.

i. Soil report from the Annapurna Hotel side is assumed to represent the property of Phora premises.

ii. The contribution of loose soil towards the Annapurna’s side from Phora Durbar is neglected during the calculation of the depth of steel pipe.
CHAPTER 3. BILLING OF QUANTITY

Detailed cost estimate and BoQ for three options is shown in the Annex-I below.
CHAPTER 4. CONCLUSION AND RECOMMENDATION

It is recommended to install single row ISNB125, 5.4 mm thickness dia steel post of 7 m length with below existing Ground level. It is recommended to demolish the remaining compound wall before construction works starts.

i. It is recommended that proper and efficient surface drainage should be provided at the location of the structures both during and after construction. Surface water should be directed away from the edges of the excavation.

ii. The materials to be used for backfilling purposes shall be of selected fill composed of sand and/or granular mixture free from organic matter or other deleterious substances. The plasticity index of the backfill material shall not exceed 10 percent. It shall be spread in lifts not exceeding 25 cm in un-compacted thickness, moisture conditioned to its optimum moisture content, and compacted to a dry density not less than 95% of the maximum dry density as obtained by modified proctor test (ASTM D-1557).

iii. The SANDY/GRAVELLY materials will probably be satisfactory for backfilling purposes, whereas the CLAYEY materials will not be satisfactory for backfilling purposes. However, the final decision shall be taken during construction after complete excavation.

iv. Superstructure is able to withstand the horizontal and vertical loads due to the wind force and the seismic loads.
ANNEX-I BILLING OF QUANTITY