DESIGN AND ANALYSIS OF UNDERGROUND AND ELEVATED SERVICE RESERVOIR IN SINGLE STRUCTURE

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Abstract - *The need for a service reservoir is as old civilization, to provide storage of water for use in many applications like drinking water, chemical manufacturing, etc. This paper presents single structured RCC Underground Service Reservoir and Elevated Service Reservoir for a capacity of 1500000 litres square shaped and 900000 litres circular shaped resp. store fully treated potable water close to the point of distribution. The main concept of designing single structured service reservoir is to increase water storage capacity and improve pressure without affecting environmental and ecological assets in the area. Also to get right water pressures to both nearby and more distant residents. Working Stress Method is used to design tank components.*

Key Words: Single structured Underground and Elevated Service Reservoir, Water storage capacity, Water pressure, Design, Working stress method

1. INTRODUCTION

Water is generally stored in concrete containers and later on, it is pumped to altered areas to serve the community. Service reservoir can be classified as overhead, resting on ground and underground depending on their location. Most water resource systems in developing countries such as India, where urbanizing is increasing day by day hence there is need to construct a greater number of service reservoirs. An elevated service reservoir is an efficient water distribution system. The basic purpose of elevated service reservoirs is to secure constant water supply with sufficient flow to wide area by gravity. The walls of underground service reservoirs are exposed to water pressure from inside and earth pressure from outside. The base of these service reservoirs is subjected to water pressure from inside and soil reaction from underneath.

1.1 OBJECTIVES

1. To carry out water demand for a selected village.
2. To design single structured underground and elevated service reservoirs.
3. To compare cost of single structured underground and elevated service reservoir with the two separate conventional service reservoir.

2. DESIGN

The village selected for the design is Hajarmachi which is located in Karad Tehsil of Satara District in Maharashtra, India.

2.1 POPULATION FORECASTING

According to census 2011 the population of Hajarmachi village is 9317.
Arithmetic method for population of year 2050 is given below.

Year	Population	Increment
2001	8330	-
2011	9317	987
2021	9607	290

Average Increment = 640

Population for year, $P_{2031} = 9607 + 640 \times 1 = 10247$

$P_{2041} = 9607 + 640 \times 2 = 10887$

$P_{2051} = 9607 + 640 \times 3 = 11527$

Daily water demand for a person per day = 135 litres

∴ Per capita water demand = Daily demand x Population

= 135 x 11527

= 1556145 litres

So, for year 2050 consider water demand is 1500000 litres

1 m^3 = 1000 litre

∴Water required = 1500 m^3

2.2 DESIGN OF ELEVATED SERVICE RESERVOIR

Consider out of full water demand, 60% of water is to be stored in elevated service reservoir and it is of circular shaped with top dome and flat bottom.

So, capacity of elevated service reservoir is 900 m^3

2.2.1 Dimensions of Elevated Service Reservoir

Let H = 4.5 m

$$\text{Volume, } V = \frac{\pi}{4} \times D^2 \times H$$

$$900 = \frac{\pi}{4} \times D^2 \times 4.5$$

$$D = 16 \text{ m}$$

Free board = 0.3 m

$\therefore$ D = 16 m and H = 4.8 m

2.2.2 Top Dome

Meridional Thrust (T_1)

$T_1 = \frac{WR}{1+\cos\theta}$

Thickness of dome = 100 mm

For W, Live Load = 1.5 KN/m^2

Self Weight = 0.10 x 25 = 2.5 KN/m^2

$\therefore$ W = 4 KN/m^2

For R, $h = \frac{1}{6} \text{ x } D = \frac{1}{6} \times 16 = 2.6$ m

$R = \frac{(D/2)^2 + h^2}{2 \times h} = \frac{(16/2)^2 + 2.6^2}{2 \times 2.6} = 13.6$ m

$\therefore T_1 = \frac{WR}{1+\cos\theta} = \frac{4 \times 13.6}{1+0.8} = 30.22$ KN/m

Meridional stress = $\frac{30.22 \times 10^2}{1000 \times 100} = 0.30$ N/mm^2

As per IS 3370 (Part 2), Permissible stress in concrete

$\therefore 0.30 < 8$ N/mm^2 ------- safe

Provide 0.3 % min reinforcement

Ast = $\frac{0.3}{100} \times 1000 \times 100 = 300$ mm^2

Provide 6 bars of 8 mm Ø @ 160 mm c/c meridionally.

Hoop Tension:

$T_2 = WR\left[\cos\theta - \frac{1}{1+\cos\theta}\right] = 4 \times 13.6\left[0.80 - \frac{1}{1+0.80}\right]$

= 13.29 KN/m

$\therefore$ Hoop stress = $\frac{13.29 \times 10^3}{1000 \times 100} = 0.13$ N/mm$^2 < 8$ N/mm2

---- safe

Provide 0.3 % min reinforcement

Ast = $\frac{0.3}{100} \times 1000 \times 100 = 300$ mm^2

Provide 6 bars of 8 mm Ø @ 160 mm c/c circumferentially.

2.2.3 Top Ring Beam

W = $T_1 \cos\theta$ = 30.22 x 0.8 = 24.176 KN/m

Total hoop tension in beam = W x $\frac{D}{2}$ = 193.36 KN

Ast for hoop tension = $\frac{T1}{\sigma st} = \frac{193.36 \times 10^3}{130} = 1487.38$ mm^2

Provide 7 bars of 16 mm Ø @135 mm c/c.

Dimensions of Ring Beam

$\sigma ct = \frac{T}{Ag+(m-1)Ast} = \frac{193.36 \times 10^3}{Ag+(9.33-1)1489.34} = 1.5$

$\therefore$ Ag = 116500 mm^2

Provide a ring beam size 350 x 350 mm

Provide 8 mmØ –2 legged vertical stirrups @ 260 mm c/c.

2.2.4 Tank Wall

Maximum hoop tension at base of wall

$T = \frac{\gamma w \times H \times D}{2} = \frac{9.81 \times 4.8 \times 16}{2} = 376.70$ KN/m

Ast = $\frac{T1}{\sigma st} = \frac{376.70 \times 10^3}{130} = 2898$ mm^2/m

Provide 10 bars of 20 mm Ø @ 100 mm ($Ast_{prov.}$ = 3141.6 mm^2)

Thickness of Wall:

$\sigma ct = \frac{T}{Ag+(m-1)Ast} = \frac{376.70 \times 10^3}{1000\, t+(9.33-1) \times 13909} = 1.5$

$\therefore$ t = 135.27 mm

Provide 200 mm thickness of wall uniform up to top of tank

Minimum reinforcement is 0.24 %

$\therefore \frac{0.24 \times 200 \times 1000}{100} = 480$ mm^2

Provide 4 bars of 12 mm Ø @230 mm c/c

Distribution steel:

Ast = $\frac{0.2 \times 200 \times 1000}{100} = 400$ mm^2

Provide 2 bars of 12 mm Ø @300 mm c/c in vertical direction.

2.2.5 Walking Gallery

Let Width of gallery = 1000 mm

Thickness of gallery = 100 mm

Live load = 1.5 KN/m

Self weight of slab = 0.1 x 1 x 25 KN/m

$\therefore$ W = 2.5 KN/m

Railing load at 1000 mm from tip = 0.80 KN/m

BM due to self weight and railing = 2.5 x $\frac{1}{2}$ + 0.8 (1-0.1) = 1.97 KN

BM due to live load (udl) = 1.5 x $\frac{1^2}{2}$ = 0.75 KN

BM due to live load (point load) = 1 x (1+0.1) = 0.9 KN

Maximum design BM, M = 1.97 + 0.75 + 0.9 = 3.62 KN

Considering, d = 100 – 30 – 4 = 66

$Ast_{req.} = \frac{M}{\sigma st \times j \times d} = \frac{3.62 \times 10^6}{130 \times 0.86 \times 66} = 490.5\ mm^2$

Provide 10 bars of 8 mm Ø @ 100 mm c/c ($Ast_{prov.}$ = 503 mm^2)

2.2.6 BASE SLAB:

Assume thickness of slab = 300 mm

Load on circular slab (W) = Weight of water + Self weight of slab

= (9.81 x 4.5) + (0.3 x 25)

W = 51.65 KN/m^2

a. Maximum radial and circumferential moments

Positive moment at centre of span (Mrp) = $\frac{wr^2}{16} = \frac{51.65\ x\ 8\ x\ 8}{16}$ = 206.6 KN/m

Negative moment at support (Mrn) = $\frac{wr^2}{8} = \frac{51.65\ x\ 8\ x\ 8}{8}$ = 413.2 KN/m

b. Circumferential moment (Mc) = $\frac{wr^2}{16} = \frac{51.65\ x\ 8\ x\ 8}{16}$ = 206.6 KN/m

Effective depth of slab = $\sqrt{\frac{206.6\ x\ 10^6}{1.8\ x\ 1000}}$ = 338.78 mm

$\therefore$ d = 350 mm

Overall depth = 380 mm

2.2.7 Bottom Ring Beam

a) Total load on ring beam

[1] Weight of water = $\gamma w \times \frac{\pi}{4} \times D^2 \times H$ = 88875.87 KN

[2] Load from Dome = $T_1 \sin\theta \times 2\pi \times D/2$ = 881.03 KN

[3] Weight of Top Ring Beam = (0.35 x 0.35) x π x 16.35 x 25 = 157.3 KN

[4] Weight of Wall = 0.2 x (4.8 – 0.35) x π x 16.2 x 25 = 1132.38 KN

[5] Weight of Bottom Slab = π x 8^2 x 0.38 x 25 = 1833.68 KN

[6] Weight of Bottom Ring Beam (Rib section 550 x 900) = 0.55 x 0.8 x π x 16.2 x 25 = 560 KN

$\therefore$ Total Vertical Load on Beam (W) = 11735.64 KN

b) Moments and Shear forces in Ring Beam

Negative BM at support = 0.0083 WR = 892.4 KN.m

Positive BM at centre of support= 0.0041 WR = 440.8KN.m

Torsional moment = 0.0006 WR = 64.5 KN.m

Shear force at support = V = $\frac{\text{Total load (W)}}{2 \times \text{No.of columns}}$ = 840 KN

Design of support section:

Effective Depth =d = $\sqrt{\frac{892.4\ x\ 10^6}{1.8\ x\ 550}}$ = 949.42 mm ≈ 950 mm

Overall Depth = D = 1000 mm

$\therefore$ Ast = $\frac{892.4 \times 10^6}{150 \times 0.87 \times 950}$ = 6714.2 mm^2

Provide 25 mm Ø bars ($Ast_{prov.}$) = 6872 mm^2)

Width of beam = 600 mm

$\tau_v = \frac{V}{bd} = \frac{840\ x\ 10^6}{600\ x\ 950}$ = 1.47 N/mm^2

$\frac{100\ Ast}{bd} = \frac{100 \times 6872}{600 \times 950}$ = 1.20

From Table of IS 456, $\tau_c = \frac{1.20-1}{1.25-1} = \frac{x-0.41}{0.45-0.41} = 0.44 \therefore \tau_c < \tau_v$

Hence shear reinforcement is required

Shear resisted by concrete = $\frac{0.44 \times 600 \times 950}{1000}$ = 250.8 KN

Balance shear = 840 – 250.8 = 589 KN

Provide 18 mmØ, two legged stirrups at 120 mm c/c.

Design of centre of span

Moment = M = 441 KNm

Ast = $\frac{441 \times 10^6}{190 \times 0.89 \times 950}$ = 2745 mm^2

Minimum quantity of steel

As = $\frac{0.85\ bd}{Fy} = \frac{0.85 \times 600 \times 950}{415}$ = 1167.46 mm^2

Provide 4 bars of 20 mm Ø bars (A_{st} = 1256 mm^2)

Design of section subjected to maximum torsion and shear

Torsion, T = 64.5 KNm

Shear Force, V = 485.33 KN

$\frac{W}{\pi D} = \frac{13440.18}{\pi \times 16.2} = 264.08$ KN/m

SF at section of max. Torsion V = 840 - $\frac{(264.08 \times 8.1 \times \pi \times 9.5)}{180}$ = 485.33 KN

Overall Depth = D = 1000 mm

Width of section = b = 600 mm

$Ms = T \times \frac{1+\frac{D}{b}}{1.7} = 101.17$ KNm

Meq. = M + Mt = 0 +101.17 = 101.17 KNm

$A_{st} = \frac{101.17 \times 10^6}{190 \times 0.89 \times 950} = 629.77$ mm^2

Min. Reinforcement = 1167.46 mm^2

Provide 4 bars of 20 mm Ø

Veq. = V + 1.6 (T/b) = 485.33 + 1.6 (64.5/0.6) = 657.33 KN

$\tau_{ve} = \frac{657.33 \times 10^3}{600 \times 950} = 1.15$ N/mm^2

$\frac{100\,Ast}{bd} = \frac{100 \times 1167}{600 \times 950} = 0.20$

$\tau_c < \tau_v$

Hence shear reinforcement is required

Using 16 mmØ, two legged stirrups with side cover of 25 mm and top and bottom cover of 50 mm.

∴ b_1 = 600 – 50 = 550 mm

d_1 = 950 – 50 = 900 mm

Provide 16 mmØ, two legged stirrups at 100 mm c/c.

2.2.8 Columns

Size of column = 450 mm x 450 mm

Total Load on each column = 1861 KN

Intensity of wind pressure 1.11 KN/m^2

Number of Columns = 8

Moment in each column at base = M= 39.88 KN.m

Provide 8 bars of 16 mm Ø and 6 mm Ø ties at 250 mm centres.

2.2.9 Bracings

Size of bracings = 400 mm x 450 mm

Number of bracings = 4

Provide 7 bars of 20 mm Ø at 135 mm c/c. and 8 mm Ø 2 legged stirrups at 250 mm c/c.

2.3 DESIGN OF UNDERGROUND SERVICE RESERVOIR

Consider full water is to be stored in underground service reservoir and it is of square shaped.

So the capacity of UGSR is 1500 m^3

2.3.1 Dimensions of Underground Service Reservoir

Let, H = 4.7 m and L = B

Volume = V = L x B x H

1500 = L x L x 4.7

L = 18 m

∴ B = 18 m

Free board = 0.3 m

∴ L = 18 m, B = 18 m and H = 5m

2.3.2 Design of Walls

i. Testing Condition:

As the tank is to be tested for leakage before filling back the surrounding soil, the only pressure acting on the tank walls is water pressure from inside.

Maximum water pressure, P = γH = 9. 81 x 5 = 49.05 KN/m^2

BM at base producing tension at water face = $\frac{P\,H^2}{15} = \frac{49.05 \times 5^2}{15}$ = 81.75 KN/m

Maximum BM causing tension away from water face = $\frac{P\,H^2}{33.5}$

= $\frac{49.05 \times 5^2}{33.5}$ = 36.60 KN/m

Thickness of Wall

Thickness of wall required for avoiding cracking can be calculated from:

$\sigma_{cbt} \times \frac{1}{6} \times b \times D^2 = M$

∴ D = 495.22 mm

Providing 500 mm thick wall

d = 500 – 40 = 460 mm

ii. Empty Tank Condition:

When tank is empty, there will be only active earth pressure acting from outside.

Active earth pressure coefficient,

$K_a = \frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1-\sin 30°}{1+\sin 30°} = 0.3$

Maximum earth pressure,

$P = K_a\,\gamma_s\,H = 0.33 \times 16.8 \times 5 = 27.72\ KN/m^2$

BM at base providing tension away from water face = $\frac{P\,H^2}{15}$ = $\frac{27.72 \times 5^2}{15}$ = 46.2 KN.m/m

Maximum BM causing tension at water face = $\frac{P\,H^2}{33.5}$ = $\frac{27.72 \times 5^2}{33.5}$ = 20.68 KN.m/m

iii. Area of Steel

Maximum BM at water face = 81.75 kN.m/m

Maximum BM away from water face = 36.60 KN.m/m

$Ast_{min.} = \frac{0.2}{100} \times 1000 \times 500 = 1000\ mm^2$

$= 500\ mm^2$ on each face

Vertical steel required at water face = $\frac{M}{\sigma st\, j\, d} = \frac{81.75 \times 10^6}{150 \times 0.87 \times 460}$ = 1361.8 mm²

Provide 7 bars of 16 mm Ø @ 145 mm c/c

Vertical steel required away from water face = $\frac{36.60 \times 10^6}{190 \times 0.89 \times 460}$ = 470.5 mm²

Provide 12 mm Ø @ 250 mm c/c horizontal bars on both faces of wall

2.3.3 Roof Slab

Moment at support = $M = \alpha_x\, w\, \ell_x^2$

$= 0.047 \times 6 \times 6.16^2 = 10.70$ KN.m

$1.65\ bd^2 = 10.76 \times 10^6$

∴ Effective depth of slab = d= 100 mm

Assuming 10 mm Ø bars and 15 mm cover

Overall Depth = D = 100 +5+15 = 120 mm

2.3.4 Beams:

Provide 450 x 450 mm continuous beams.

2.4 RAFT FOUNDATION

Total load on each column =1861 KN

Approximate weight of foundation (10% of column loads) = $14888 \times \frac{10}{100}$ =1488.8KN

∴ Total load transmitted to the soil =16376.8 KN

Safe bearing capacity of soil =130 KN/m²

∴ Area of raft foundation = $\frac{16376.8}{130}$ =125.97 ≈ 126 m²

Total length of the raft slab = (4 x 11.46 + 4 x 16.2)

=110.64 m

∴ Width required for raft = $\frac{126}{110.64}$ = 1.14 m

Provide a width of 1.2 m for the raft slab net upward pressure intensity on the raft slab = $\frac{148888}{111 \times 1.20}$ =111.77 KN/m²

2.4.1Design of raft slab

Projection of raft slab from the face of raft beam = $\frac{1.2-0.45}{2}$ = 0.375 m

Consider a 1 meter wide strip of the rat slab cantilever from the face of the beam

Max. Bending moment = $\frac{wl^2}{2} = \frac{111.7 \times (0.375)^2}{2}$ = 8 KN.m

Factored moment, $M_u = 1.5 \times 8 = 12$ KN.m

$0.138\ F_{ck}\ bd^2 = 0.138 \times 1000 \times d^2 = 12 \times 10^6$

∴ d = 53.83 mm

Overall depth =53.83 + 50 = 103.83 mm

Provide overall depth of 150 mm

Actual effective depth = d = 150 - 50 =150 mm

$\frac{Mu}{bd^2} = \frac{12 \times 10^6}{1000 \times 100^2} = 1.2$

Percentage of steel required

$$Pt = 50\left[\frac{1-\sqrt{1-\frac{4.6}{fck}\times Mu}}{\frac{fy}{fck}}\right] = 50\left[\frac{1-\sqrt{1-\frac{4.6}{30}\times 1.2}}{\frac{415}{30}}\right] = 0.35\ \%$$

$Ast = \frac{pt}{100}\,bd = \frac{0.35}{100} \times 1000 \times 100 = 350\ mm^2$

Provide 5 bars of 10 mm Ø @ 220 mm c/c (Ast pro. = 357 mm²)

2.4.2 Design of continuous raft beam

Upward load transmitted to the beam per Meter = 112 x 1.2 = 134.4 KN/m²

Case 1

$$\text{Max. BM} = \frac{wl^2}{10} = \frac{134.4 \times (16.6)^2}{10} = 3703.5 \text{ KN.m}$$

Mu = 1.5 x 3703.5 = 5555.25 KN.m

$5555.25 \times 10^6 = 0.138 \times 30 \times 450 \times d^2$

$\therefore$ d = 1726.8 mm ≈ 1730 mm

Provide 20 mm Ø bars

Overall thickness of beam = 1792 ≈ 1800mm

$\therefore$ Effective depth = d =1800 – 65 =1735 mm

$$\frac{Mu}{bd^2} = \frac{5555.25 \times 10^6}{450 \times 1735^2} = 4.10$$

$$P_t = 50\left[\frac{1-\sqrt{1-\frac{4.6}{fck}\times Mu}}{\frac{fy}{fck}}\right] = 50\left[\frac{1-\sqrt{1-\frac{4.6}{30}\times 4.10}}{\frac{415}{30}}\right] = 1.41\ \%$$

$$Ast = \frac{pt}{100} \times bd = \frac{1.41}{100} \times 450 \times 1735 = 11008.5 \text{ mm}^2$$

Design of Shear

Max. shear force = 0.6 wl = 1338.6 KN

Factored SF = V_u = 1.5 x 1338.6 = 2007.9 KN

$$\tau_v = \frac{2007.9 \times 10^3}{450 \times 1735} = 2.57 \text{ N/mm}^2$$

$$P_t = \frac{Ast}{bd} \times 100 = \frac{11259}{450 \times 1735} \times 100 = 1.48\ \%$$

For 1.44 % steel, τ_c =0.748 N.mm²

Shear resistance of concrete

τ_c bd = 0.748 x 450 x 1735 = 584001 N

Net shear = 2007900 - 584001= 1423899 N

Provide 4 legged 10 mm Ø stirrups @135 mm c/c

Case 2:-

$$\text{Max. BM} = \frac{wl^2}{10} = \frac{134.4 \times (11.76)^2}{10} = 1858.71 \text{ KN.m}$$

Mu = 1.5 x 1858.71 = 2788.065 kN.m

$2788.06 \times 10^6 = 0.138 \times 30 \times 450 \times d^2$

$\therefore$ d= 1223.33 mm

Overall thickness of beam = 1288.33 ≈ 1300 mm

Effective depth = d = 1300 – 65 = 1235 mm

$$\frac{Mu}{bd^2} = \frac{2788.06 \times 10^6}{450 \times (1235)^2} = 4.06$$

$$Pt = 50\left[\frac{1-\sqrt{1-\frac{4.6}{30}\times 4.06}}{\frac{415}{30}}\right] = 1.39\ \%$$

$$Ast = \frac{pt}{100} \times bd = \frac{1.39}{100} \times 450 \times 1197 = 7724.9 \text{ mm}^2$$

Provide 32 mm Ø bars at 100 mm c/c.

Design of Shear

Max. SF = 0.6wl = 0.6 x 134.4 x 11.76 = 948.32 KN

Factored Sf = Vu = 1.5 x 948.32 = 1422.48 KN

$$\tau_v = \frac{1422.48 \times 10^3}{450 \times 1235} = 2.55 \text{ N/mm}^2$$

$$Pt = \frac{Ast \times 100}{bd} = \frac{8042.47 \times 100}{450 \times 1197} = 1.44\ \%$$

For 1.44 % steel, τ_c = 0.748 N/mm²

Shear resistance of concrete ($\tau_c < \tau_v$)

τ_c bd = 0.748 x 450 x 1235 = 415701 N

Net Shear = 1422480 – 415701 = 1006779 N

Provide 4 legged 10 mm Ø stirrups @ 135 mm c/c.

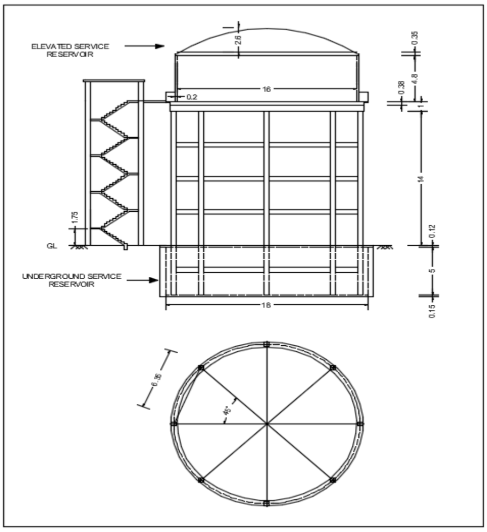

Fig No -1: Single Structured UGSR and ESR

2.5 COST

Estimated cost for single structured Underground and Elevated Service Reservoir is Rs.1,17,00,000 and total cost of two separate conventional underground service reservoir and elevated service reservoir is Rs.1,14,00,000.

3. CONCLUSIONS

The design of single structured ESR and UGSR is designed manually and a rough estimation for the proposed service reservoir is included.

1. Per capita demand has been calculated which helped us, to know about the water consumption in Hajarmachi village and further helped in design the service reservoir.
2. ESR provide head for supply of water. When water has to be pumped into the distribution system at high heads without any pumps for supply however pumps are necessary for pumping only till tank is filled, once it is stored in tank the gravity creates the pressure for free, unlike pumps. We need pressurized water to fledge and make taps eject water at an appropriate rate. Elevated tanks do not require continuous operation of pump, as it will not affect the distribution system since the pressure is maintained by gravity.
3. The design of two separate reservoirs in a single structure is space saving, high storage capacity of water and cost is estimated nearly equal when compared to cost of two separate conventional service reservoirs.

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