Design of reinforced concrete tower (Draft)

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1 Receiver mass

The mass of the sodium receiver is obtained from adding the mass of panels, structure of support and heat transfer fluid (HTF) as in Eq. (1). Each mass is calculated individually based on the receiver thermal output, operation temperatures, and geometry.

$$m_{\mathrm{rec}} = m_{\mathrm{scaffholding}} + m_{\mathrm{panels}} + m_{\mathrm{installation}} + m_{\mathrm{sodium}} \tag{1}$$

1.1 Scaffholding and installation

The weight of the structure of support (scaffholding and installation) is scaled to be linear-proportional to the volume of the receiver

$$m_{\mathrm{scaffholding}} = m_{\mathrm{scaffholding,ref}} \left(\frac{V_{\mathrm{rec}}}{V_{\mathrm{rec,ref}}} \right) \tag{2}$$

$$m_{\mathrm{installation}} = m_{\mathrm{installation,ref}} \left(\frac{V_{\mathrm{rec}}}{V_{\mathrm{rec,ref}}} \right) \tag{3}$$

where V_{rec} is the volume of the sodium receiver, and $V_{\mathrm{rec,ref}}$ is the volume of the CMI-Atacama receiver. The volume of receiver is calculated as

$$V_{\mathrm{rec}} = \frac{\pi}{4} \cdot D_{\mathrm{rec}}^2 \cdot H_{\mathrm{rec}} \tag{4}$$

1.2 Panels

The weight of the receiver panels is scaled to be linear-proportional to the area of panels

$$m_{\mathrm{panels}} = m_{\mathrm{panels,ref}} \left(\frac{A_{\mathrm{rec}}}{A_{\mathrm{rec,ref}}} \right) \tag{5}$$

where A_{rec} is the panel area of the sodium receiver, and $A_{\mathrm{rec,ref}}$ is the panel area of the CMI-Atacama receiver. The area of panels is calculated as

$$A_{\mathrm{rec}} = \pi \cdot D_{\mathrm{rec}} \cdot H_{\mathrm{rec}} \tag{6}$$

1.3 Sodium

The weight of the heat transfer fluid is scaled based on the assumption that each receiver (sodium or salt) will require the same time to be filled, therefore

$$\frac{m_{\mathrm{sodium}}}{\dot{m}_{\mathrm{sodium}}} = \frac{m_{\mathrm{salt}}}{\dot{m}_{\mathrm{salt}}} \tag{7}$$

but mass flow rates are computed from receiver capacities and enthalpy difference, then

$$\frac{m_{\mathrm{sodium}} \Delta h_{\mathrm{sodium}}}{\dot{Q}_{\mathrm{rec}}} = \frac{m_{\mathrm{salt}} \Delta h_{\mathrm{salt}}}{\dot{Q}_{\mathrm{rec,ref}}} \tag{8}$$

Finally, solving for m_{sodium} the following equation is obtained

$$m_{\text{sodium}} = m_{\text{salt}} \left(\frac{\dot{Q}_{\text{rec}}}{\dot{Q}_{\text{rec,ref}}} \right) \left(\frac{\Delta h_{\text{salt}}}{\Delta h_{\text{sodium}}} \right) \tag{9}$$

where $\dot{Q}_{\text{rec}}$ is the thermal output at design of the sodium receiver, and $\dot{Q}_{\text{rec,ref}}$ is the thermal output at design of the CMI-Atacama receiver (salt as HTF). The thermo-physical properties of salt are calculated based on correlations developed by Zavoico [1], while the thermo-physical properties of liquid sodium are computed based on correlations developed by Fink and Leibowitz [2].

1.4 Mass breakdown CMI receiver

The CMI receiver has a thermal capacity at design of 700 MWh_{t}, and its dimensions are $D = 17$ m and $H = 18$ m. The operation temperatures for the HTF are 290°C at the receiver inlet and 566°C at the outlet. Table shows the mass breakdown for this receiver, whichi were obtained from Burghartz [3].

Table 1: Mass breakdown of CMI receiver [3].

Item	Unit	Value
Scaffholding	metric tons	1000.0
Installation	metric tons	650.0
Panels	metric tons	100.0
HTF (salt)	metric tons	500.0

2 Design loads

2.1 Wind loads

The reinforced concrete structure must resist the wind forces in the along-wind direction. In addition, the hollow circular cross section must resist the loads due to the circumferential pressure distribution. The procedure to determine wind loads follows the guidelines of the ACI 307 from the American Concrete Institute [4].

2.1.1 Design wind speed

To determine wind loads, the height of the structure is divided into n sections as depicted in Figure (1). The wind loads are calculated at the height to the middle of each section (h_j).

The mean hourly design speed $\bar{V}_j$, in ft/s, at a height h_j above the ground is computed from

$$\bar{V}_j = 0.9555 \cdot V_r \left(\frac{h_j}{33} \right)^{0.154} \tag{10}$$

where the reference design wind speed (V_r) is be the 3-second gust wind speed at 33 ft over open terrain, which is calculated as

$$V_r = (I)^{0.5} V \tag{11}$$

where V is the basic wind speed, in mph, obtained from ASCE 7. A value of $V = 97$ mph was obtained for the location of the structure (Daggett, USA) considering risk category II, due to the low occupancies associated with power tower installations as defined in section 1.5.1 of ASCE 7 [5].

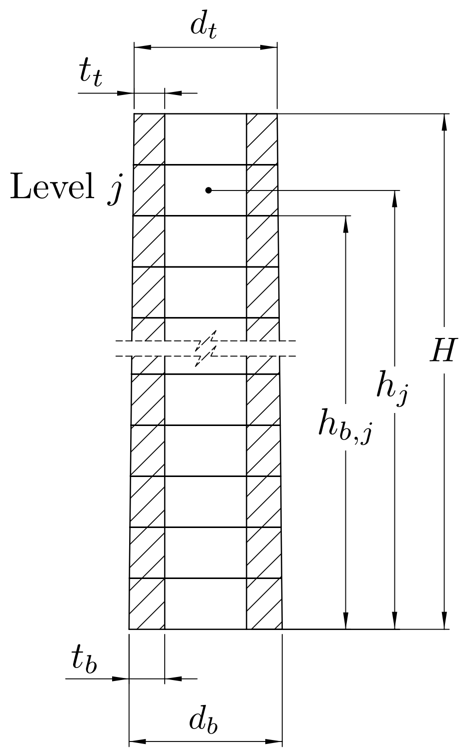

Figure 1: Sections of the structure.

2.1.2 Along-wind loads

The along-wind load per unit length ($w_{w,j}$) is be sum of the mean load ($\bar{w}_j$) and the fluctuating load (w'_j). The mean load is computed from

$$\bar{w}_j = C_{dr,j} \cdot d_j \cdot \bar{p}_j \tag{12}$$

where $C_{dr,j}$ is the drag coefficient at section j above the ground and $\bar{p}_j$ is the pressure due to mean hourly design wind speed. The drag coefficient is obtained as

$$C_{dr,j} = \begin{cases} 0.65, & h_j < H - 1.5d_t \\ 1.0, & h_j \geq H - 1.5d_t \end{cases} \tag{13}$$

where H is the height of the structure and d_t is the diameter at the top. The pressure due to mean hourly design wind speed is computed from

$$\bar{p}_j = 0.00119 \cdot K_d \cdot \bar{V}_j^2 \tag{14}$$

where K_d is a geometry factor for circular shapes and its value is 0.95.

The mean shear force $\bar{F}_{v,j}$ and bending moment $\bar{M}_j$ at the base of each section j are obtained from Eqs. (15) and (16) as a function of the mean load $\bar{w}_i$ and section height Δz.

$$\bar{F}_{v,j} = \sum_{i=j}^{n} \bar{w}_i \cdot \Delta z \tag{15}$$

$$\bar{M}_j = \sum_{i=j}^{n} \bar{w}_i \cdot (h_i - h_{b,j})\, \Delta z \tag{16}$$

The fluctuating load w'_j, in lb/ft, is computed from

$$w'_j = \frac{3.0 \cdot h_j \cdot G_{w'} \cdot \bar{M}_b}{H^3} \tag{17}$$

where $G_{w'}$ is the gust factor and $\bar{M}_b$ is the bending moment at the base of the structure due to mean hourly design wind speed. The gust factor is obtained from

$$G_{w'} = 0.30 + \frac{11.0\left[T \cdot \bar{V}(33)\right]^{0.47}}{(h+16)^{0.86}} \tag{18}$$

where $\bar{V}(33)$ is the mean hourly speed at 33 ft, and T is the natural fundamental period. T is obtained from Eq. (19) as a function of the geometrical parameters from Figure (1), the density of concrete (ρ_{ck}), and the modulus of elasticity of concrete (E_{ck}).

$$T = 5\left(\frac{h^2}{d_b}\right)\sqrt{\frac{\rho_{ck}}{E_{ck}}}\left(\frac{t_t}{t_b}\right)^{0.3} \tag{19}$$

The fluctuating shear force $F'_{v,j}$ and bending moment M'_j are obtained from the mean load as in Eqs. (15) and (16).

$$F'_{v,j} = \sum_{i=j}^{n} w'_i \cdot \Delta z \tag{20}$$

$$M'_j = \sum_{i=j}^{n} w'_i \cdot (h_i - h_{b,j})\,\Delta z \tag{21}$$

Finally, the total bending moment $M_{w,j}$ due to along-wind loads for section j is computed as

$$M_{w,j} = M'_j + \bar{M}_j \tag{22}$$

2.1.3 Circumferential wind loads

The maximum circumferential bending moment due to the radial wind pressure is computed by Eqs. (23) and (24)

$$M_{i,j} = 0.31 \cdot G_{r,j} \cdot \bar{p}_j \cdot r_j^2 \tag{23}$$

$$M_{o,j} = 0.27 \cdot G_{r,j} \cdot \bar{p}_j \cdot r_j^2 \tag{24}$$

where $G_{r,j}$ is the gust factor for radial wind pressure, which is calculated from Eq. (25).

$$G_{r,j} = \begin{cases} 4.0 - 0.8\log_{10}(h_j), & h_j < 1.0 \text{ ft} \\ 4.0, & h_j \geq 1.0 \text{ ft} \end{cases} \tag{25}$$

2.2 Seismic loads

The ACI 307-08 recommends that earthquake loads shall be determined by means of the dynamic response spectrum analysis method from the ASCE 7. The equations within this section follows *Equivalent lateral force procedure* described in section 12.8 of the ASCE 7 [5].

2.2.1 Site-dependent seismic data

The mapped maximum considered earthquake (MCE) spectral response acceleration at short periods (S_S) and at 1 second (S_1) are obtained from the Applied Technology Council (ATC) hazards website[1] for the specific location of the structure (Daggett, USA). Table 2 shows the MCE spectral response acceleration data for this study, considering risk category II and site class as D[2].

The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1 second (SM_1), adjusted for site class effects, is be determined as

$$S_{MS} = F_a S_S \tag{26}$$

[1] https://hazards.atcouncil.org/

[2] According to ACI 307 and ASCE 7, when soil properties are not known in sufficient detail to determine the site class, class D shall be used

Table 2: Site-dependent seisimic data for Dagget (USA).

Item	Value
Latitude (+N)	34.86
Longitude (+E)	-116.89
S_S	1.307
S_1	0.467

$$S_{M1} = F_V S_1 \tag{27}$$

where the site dependent coefficients F_a and F_V shall be obtained from Tables (3) and (4).

Table 3: Short-period site coefficient, F_a

Site class	$S_S < 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.8	0.8
C	1.3	1.3	1.2	1.2	0.8	0.8
D	1.6	1.4	1.2	1.1	0.8	0.8
E	2.4	1.7	1.3	See section 11.4.8	See section 11.4.8	See section 11.4.8
F	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_S.

Table 4: Long-period site coefficient, F_V

Site class	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2	2.0	1.9	1.8	1.7
E	4.2	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8
F	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8	See section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.

The design earthquake spectral response acceleration at short periods (S_{DS}) and at 1 second (S_{D1}), is be determined as

$$S_{DS} = (2/3) S_{MS} \tag{28}$$

$$S_{D1} = (2/3) S_{M1} \tag{29}$$

2.2.2 Seismic base shear

The seismic base shear, F_V, in a given direction shall be determined as

$$F_V = C_s W \tag{30}$$

where C_s is the seismic response coefficient, which is obtained from Eq. (31), and W is the weight of the structure. The seismic importance factor $I_e = 1.0$ is selected due to the low occupancies associated with power tower installations as defined in section 1.5.1 of ASCE 7 [5].

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{31}$$

The value of C_s shall not exceed the upper limit from Eq. (32):

$$C_{s,\max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \tag{32}$$

where T is the natural period, which is obtained from Eq. (19).

2.2.3 Vertical distribution of seismic forces

The lateral seismic force (F_j) induced at any section defined in Figure (1), is determined from the seismic base shear force (F_V) as

$$F_j = C_{v,j} \cdot F_V \tag{33}$$

where the vertical distribution factor $C_{v,j}$ at section j. The vertical distribution factor is calculated as

$$C_{v,j} = \frac{W_j \cdot h_{b,j}^k}{\sum_{i=1}^{n} W_i \cdot h_{b,i}^k} \tag{34}$$

where W_j and W_j are the portion of the total effective seismic weight of the structure located or assigned to sections i or j; and k is an exponent related to the structure period as follows:

- For structures that have a period of 0.5 s or less, $k = 1$.
- For structures that have a period of 2.5 s or more, $k = 2$.
- For structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1.0 and 2.0.

The seismic design story shear in any section ($F_{v,j}$) shall be determined from the following equation:

$$F_{v,j} = \sum_{i=j}^{n} F_i \tag{35}$$

and the bending moment is obtained from the lateral seismic force and the section-dependent numerical coefficient from the ACI 307 [6]:

$$M_{s,j} = \sum_{i=j}^{n} J_j \cdot F_i(h_i - h_{b,j}) \tag{36}$$

where the section-dependent numerical coefficient for base moment, J_j is calculated from Eq. (37).

$$J_j = J + (1 - J)\left(\frac{h_{b,j}}{h}\right) \tag{37}$$

The total numerical coefficient for base moment J is calculated from Eq. (38) as a function of the fundamental period.

$$J = \begin{cases} 0.6/\sqrt[3]{T}, & 27/125 < T < 64/27 \\ 0.45, & T \geq 64/27 \\ 1.0, & T \leq 27/125 \end{cases} \tag{38}$$

3 Combination of loads

3.1 Vertical loads

At any section, the ACI 307 requires to determine the strength U_v to resist dead load D, wind load W or seismic load E. U_v should be the largest of the following [4]:

$$U_v = 1.4D \tag{39}$$

$$U_v = 1.2D + 1.6W \tag{40}$$

$$U_v = 1.2D + 1.0E \tag{41}$$

where

$$D = w_j \tag{42}$$

$$E = F_{v,j} \tag{43}$$

$$W = \bar{F}_{v,j} + F'_{v,j} \tag{44}$$

3.2 Circumferencial loads

The required circumferential strength U_c should be

$$U_c = 1.4W_c \tag{45}$$

where W_c is the maximum circumferential moment of each section.

3.3 Bending moment

The factored bending moment M_u at any section j is obtained as

$$M_u = M_{s,j} + M_{w,j} \tag{46}$$

4 Nominal moment strength

4.1 Vertical reinforcement

The ACI 307-08 uses ultimate strength design (USD) to calculate the nominal moment strength for each section of the structure. The deduction of this method is presented in Appendix A of ACI 307 [4].

The nominal moment strength M_n for a section j is calculated as

$$M_n = \phi K_3 \cdot P_u \cdot r \tag{47}$$

where r is the average section radius at any section, ϕ is a strength reduction factor, P_u is the factored load as defined in section 3, and K_3 is a geometrical variable calculated as

$$K_3 = \cos(\alpha) + \frac{K_2}{K_1} \tag{48}$$

where K_1 and K_2 are obtained from the reinforcement ratio ρ_t, the isotropic yield strength for structural steel f_y, the modulus of elasticity of steel E_s, and the compressive strength of concrete f'_c by solving the following equations:

$$K_1 = 1.7Q\lambda + 2\epsilon_m K_e \omega_t Q_1 + 2\omega_t \lambda_1 \tag{49}$$

$$K_2 = 1.7Q\bar{R} + \epsilon_m K_e \omega_t Q_2 + 2\omega_t K \tag{50}$$

$$K = \sin\psi + \sin\mu + (\pi - \psi - \mu)\cos\alpha \tag{51}$$

$$K_e = \frac{E_s}{f_y} \tag{52}$$

$$\omega_t = \frac{\rho_t f_y}{f'_c} \tag{53}$$

$$\lambda = \tau - n_1 \beta_1 \tag{54}$$

$$\lambda_1 = \mu + \psi - \pi \tag{55}$$

$$\bar{R} = \sin\tau - \lambda\cos\alpha - \left(\frac{n_1}{2}\right)[\sin(\gamma+\beta) - \sin(\gamma-\beta)] \tag{56}$$

$$Q_1 = \frac{\sin\psi - \sin\mu - (\psi-\mu)\cos\alpha}{1-\cos\alpha} \tag{57}$$

$$Q_2 = \frac{(\psi-\mu)(1+2\cos^2\alpha) + (1/2)(4\sin 2\alpha + \sin 2\psi - \sin 2\mu) - 4\cos\alpha(\sin\alpha + \sin\psi - \sin\mu)}{1-\cos\alpha} \tag{58}$$

where n_1 is the number of openings entirely in the compression zone. For no openings, $n_1 = \gamma = \beta = 0$. The angles β, α, μ, τ, and ψ are shown in Fig. 5.1(a) and are computed from

$$\cos\tau = a - \beta_1(1-\cos\alpha) \tag{59}$$

$$\cos\psi = \cos\alpha - \left(\frac{1-\cos\alpha}{\epsilon_m}\right)\left(\frac{f_y}{E_s}\right) \geq -1.0 \tag{60}$$

$$\cos\mu = \cos\alpha + \left(\frac{1-\cos\alpha}{\epsilon_m}\right)\left(\frac{f_y}{E_s}\right) < 1.0 \tag{61}$$

The strength factor β_1 is calculated from the compressive strength of concrete (f'_c) as

$$\beta_1 = \begin{cases} 0.85, & f'_c \leq 4 \text{ ksi} \\ 0.85 - 0.05(f'_c - 4), & 4 \text{ ksi} < f'_c \leq 8 \text{ ksi} \\ 0.65, & 8 \text{ ksi} < f'_c \end{cases} \tag{62}$$

and the maximum concrete compressive strain ϵ_m is computed from Eq. (63).

$$\epsilon_m = 0.07\left(\frac{1-\cos\alpha}{1+\cos\alpha}\right) \leq 0.003 \tag{63}$$

The stress correction factor Q is computed from

$$Q = \begin{cases} (-0.523 + 0.181\alpha - 0.0154\alpha^2) + (41.3 - 13.2\alpha + 1.32\alpha^2)\left(\frac{t}{r}\right) & \alpha \leq 5^\circ \\ (-0.154 + 0.01773\alpha + 0.00249\alpha^2) + (16.42 - 1.980\alpha + 0.0674\alpha^2)\left(\frac{t}{r}\right) & 5 < \alpha \leq 10^\circ \\ (-0.488 + 0.076\alpha) + (9.758 - 0.640\alpha)\left(\frac{t}{r}\right) & 10 < \alpha \leq 17^\circ \\ (-1.345 + 0.2018\alpha + 0.004434\alpha^2) + (15.83 - 1.676\alpha + 0.03994\alpha^2)\left(\frac{t}{r}\right) & 17 < \alpha \leq 25^\circ \\ (0.993 - 0.00258\alpha) + (-3.27 + 0.0862\alpha)\left(\frac{t}{r}\right) & 25 < \alpha \leq 35^\circ \\ 0.89 & 35 < \alpha \end{cases} \tag{64}$$

In addition, the method requires that

$$K_1 = \frac{P_u}{r \cdot t \cdot f'_c} \tag{65}$$

where t is the average thickness of the section.

The solution method proposed by the ACI 307-08 is to assume a value for ρ_t, find the value of M_n, and compare it with the value of M_u as defined in section 3. If $M_n < M_u$ then ρ_t should be increased until $M_n \geq M_u$.

4.2 Circumferential reinforcement

Any horizontal strip of the concrete column shall be designed as a horizontal beam resisting circumferential bending moments. Therefore, the circumferential strength $M_{n,c}$ is obtained as

$$M_{n,c} = A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c \cdot b_w} \right) \tag{66}$$

where the width of the section d is obtained from the minimum thickness t_{min} and the radius of the reinforcing steel bar r_s as

$$d = t - t_{\text{min}} - r_s \tag{67}$$

The height of the section b_w is assumed as 1 ft to obtain the required reinforcement ratio per unit length of height, and the strength factor β_1 is calculated from Eq. (62) as a function of the concrete strength f'_c.

As in section 4.1, the value of ρ_t is adjusted until $M_{n,c} \geq U_c$

4.3 Special design considerations and requirements

According to the ACI 307-08, two layers of vertical and circumferential reinforcement are required. In addition, the design must meet the following requirements:

- The total vertical reinforcement shall be not less than 0.25% of the concrete area.
- The outside vertical reinforcement shall be not less than 50% of the total vertical reinforcement.
- Vertical reinforcing bars shall not be smaller than US No. 4.
- The total circumferential reinforcement shall not be less than 0.20% of the concrete area.
- The circumferential reinforcement in each face shall be not less than 0.1% of the concrete area at the section.

5 Cost of the structure

The cost of the structure is computed from the cost of steel C_{steel}, concrete C_{concrete} and a fixed cost C_0 in Eq. (68). The costing assumptions are explained below and the flowchart for the design process is depicted in Appendix A.

$$C_{\text{tower}} = C_0 + C_{\text{steel}} + C_{\text{concrete}} \tag{68}$$

5.1 Cost of steel

The cost of steel C_{steel} is obtained from

$$C_{\text{steel}} = \rho_s c_{\text{steel}} \sum_{j=1}^{n} \left[\rho_{t,v} A_j \Delta z + \rho_{t,c} \pi A_c d_j \right] \tag{69}$$

where ρ_s is the density of steel, c_{steel} is the unit cost of steel (in USD/ton), A_j is the cross-section area for vertical loads at section j, A_c is the cross-section area for circumferential loads at section j, and d_j is the outer diameter at section j.

5.2 Cost of concrete

The cost of concrete C_{concrete} is obtained from

$$C_{\text{concrete}} = (1.1)\rho_{ck} \cdot c_{\text{concrete}} \cdot v_{\text{tower}} \tag{70}$$

where ρ_{ck} is the density of concrete, c_{concrete} is the unit cost of steel (in USD/CY), and v_{tower} is volume of the tower. The volume of concrete is increased in 10% to account for uncertainties.

5.3 Fixed cost

The fixed cost account for the cost of embedded metals, foundation and sitework.

5.4 Costing assumptions

Table 5 shows the costing data for steel, concrete, foundation, embedded metals, and sitework for a 268m high reinforced concrete tower published in 2010 by Kelly [7]. The costing data was scaled to 2020 USD using the chemical engineering plant cost index (CEPCI) [8].

Table 5: Costing data for a 268m high reinforced concrete tower from Kelly [7].

Item	Unit	Value 2010
Foundation	USD	18,137,087.0
Sitework	USD	996,044.0
Tower embedded metals	USD	455,770.0
Concrete	USD/CY	2,364.3
Steel	USD/ton	421.0

6 Implementation in Python

The steps described in this document are implemented in Python and can be downloaded from:

- `https://github.com/arfontalvoANU/ConcreteTower`

References

[1] A. B. Zavoico, "Solar power tower design basis document," tech. rep., Sandia National Labs., 2001.

[2] J. Fink and L. Leibowitz, "Thermodynamic and transport properties of sodium liquid and vapor," tech. rep., Argonne National Lab., 1995.

[3] A. K. Burghartz, "Investigation of selected tower construction methods for solar power plants under consideration of the receiver mounting," Master's thesis, Dissertation (Master) - ITA, 2016.

[4] A. C. 307, "Code requirements for reinforced concrete chimneys and commentary," American Concrete Institute, 2008.

[5] S. E. Institute, "Minimum design loads for buildings and other structures," American Society of Civil Engineers, 2016.

[6] A. C. 307, "Code requirements for reinforced concrete chimneys and commentary," American Concrete Institute, 1979.

[7] B. D. Kelly, "Advanced thermal storage for central receivers with supercritical coolants," tech. rep., Abengoa Solar Inc., 2010.

[8] E. Indicators, "Chemical engineering plant cost index," *Chemical Engineering*, vol. 128, p. 48, 2021.

A Flowchart for the design of the reinforced concrete tower

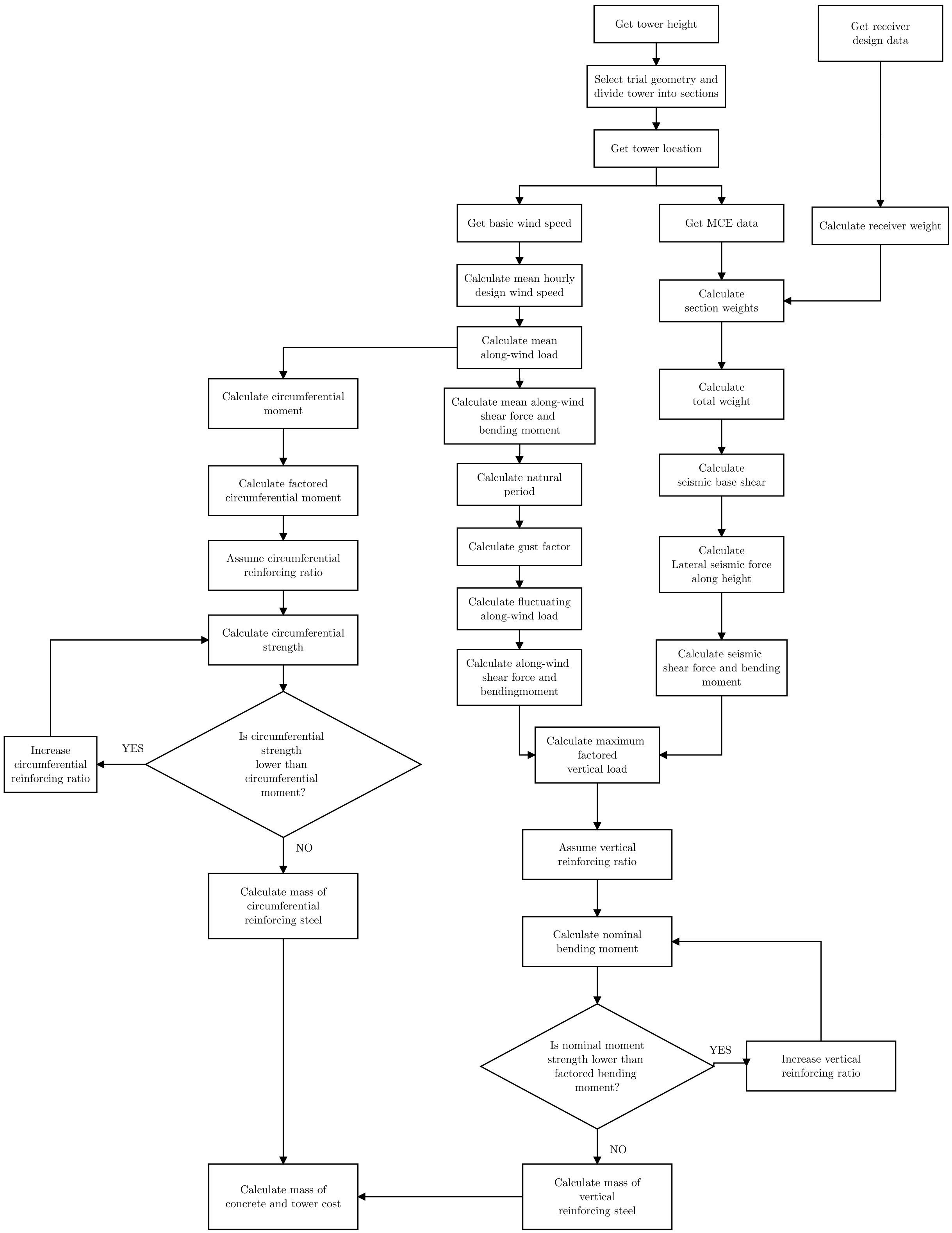

Figure 2: Sequence for tower design.