CHAPTER II
LITERATURE REVIEW

SNI 03-1726-2002 classifies the performance based design at the time receiving the earthquake loads as follow:

1. Due to small earthquake (50 years return period), the building structure may be damaged, either the structural elements or the nonstructural elements.
2. Due to medium earthquake (75 years return period), structural elements must not be damaged; the nonstructural elements may have minor damage.
3. Due to large earthquake (500 years return period), both structural elements and nonstructural elements damage, but the building must not collapse.

Moment resistance frame is the system which the structural elements (including beam, column, and joints) restrain the force due to flexural, shear and axial force. SNI-03-1726-2002 differentiates between three types of moment resisting frame:

1. Ordinary Moment Resisting Frame; comply with the provision in section 3 up to section 20 of the SNI 03-2847-2002. Basically this moment resisting frame does not meet the special detailing requirements for ductile behavior and compatible in low risk seismic zones (Zone 1 and 2)
2. Intermediate Moment Resisting Frame (IMRF); must comply the provision as ordinary moment resisting frame including section 23.2(2(3)) and 23.10. This moment resisting frame have a medium ductility and compatible in seismic zone 1-4
3. Special Moment Resisting Frame (SMRF); comply the provision as ordinary moment resisting frame including section 23.2 to 23.5. This frame has a fully ductility and use in Zone 5 and 6 which is high risk seismic zones.

2.1. Basic Design Theory

2.1.1. Loading Analysis

According to nature and source, the loads are classified into three classes: dead loads due to the weight of the structural system itself and any other material permanently attached to it, live loads which are moveable or moving load due to the use of the structure, and environmental loads which are caused by environmental effects, such as wind, snow, and earthquakes.(Kassimali, 2005)

The load specified in the building codes are considered as service load (unfactored load). Factored load are service load multiplied by the appropriate load factors specified for required strength. According to SNI 03-2847, required strength to resist loading combinations (factored loads) are as follows:

$U = 1.4D$ ……………………………………………………………(2.1)

$U = 1.2D+1.6L$ ……………………………………………………(2.2)

If the earthquake effects are considered:

$U = 1.2D + 1L \pm 1Ex \pm 0.3Ey$ ………………………………………(2.3)

$U = 1.2D + 1L \pm 0.3Ex \pm 1Ey$ ………………………………………(2.4)

$U = 0.9D \pm 1Ex \pm 0.3Ey$ ……………………………………………(2.5)

$U = 0.9D \pm 0.3Ex \pm 1Ey$ ……………………………………………(2.6)

where: D = dead load

L = Live load

E =earthquake load

The design strength is computed with input the strength reduction factor (φ) based on SNI 03-2847-2002 section 11.3 are as follow:

1. Flexure sections without axial load = 0.80
2. Axial load and axial load with flexure
   a. Axial tension and axial tension with flexure = 0.80
   b. Axial compression and axial compression with flexure
      i. Structural component with spiral reinforcement = 0.70
      ii. Other structural component = 0.65
3. Shear and torsion = 0.75
4. Concrete support = 0.65

5. Structural plain concrete = 0.55

2.1.2. Seismic Design

Earthquake resistance design is generally planned through nominal earthquake load (V_n). V_n is the result of elastic earthquake load (V_e) devided by structural response modification factor (R).

$V_e = C_1 I W_t$ and $V_n = \frac{V_e}{R}$

Therefore,

$$V_n = \frac{C_1 I}{R} W_t \quad \text{……………………………………………………..} \quad (2.7)$$

where,

V_n = horizontal base shear force as the dynamic first mode response

C_1 = base shear coefficient

I = building type factor

R = response reduction factor

W_t = total weight of the building, including live load that is considered as fixed as big as 25% to 30% of total live

Based on SNI 03-1726-2002 section 5.6, the limitation of natural fundamental period used to prevent the structure to become too flexible.

$T_1 < \zeta\, n$ …………………………………………………..………………………… (2.8)

with:

ζ = coefficient for earthquake zone which depend on the structure's location

n = number of story

2.1.3. Beam Design

2.1.3.1. The design of the flexure reinforcement

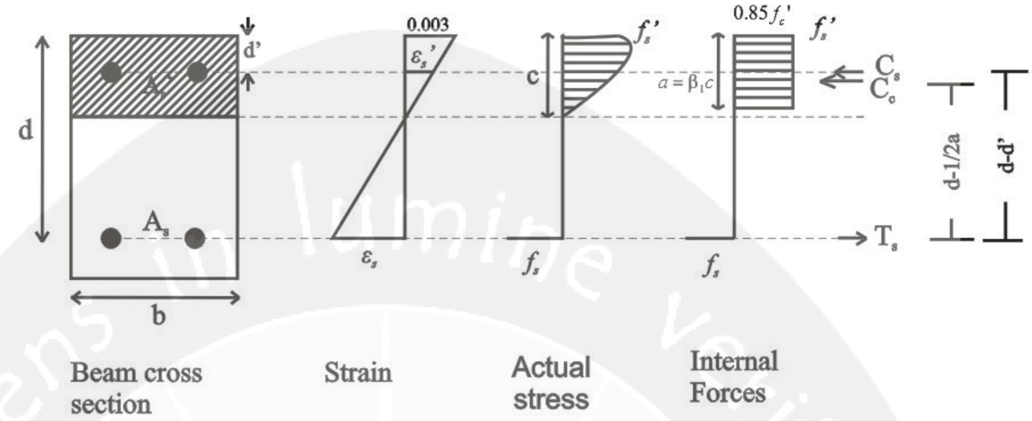

Figure 2.1 Diagram of stress in beam

Concrete compression force:

$C_c = 0.85\ f_c'\ a\ b$ ………………………………………………………….. (2.9)

Steel reinforcement compression force:

$C_s = A_s'\ f_s'$ ……………………………………………………………(2.10)

Steel reinforcement tension force:

$T_s = A_s\ f_y$ ....(2.11)

The procedure to compute the flexure strength of doubly reinforced sections can be performed as follows:

1. Assume tension steel yields
2. Check if the compression steel yields using:

$$(\rho - \rho') \leq \frac{0.85.f_c'.\beta_1.d'}{f_y.d}.\frac{600}{(600 - f_y)} \quad \text{……………………………… (2.12)}$$

3. If the compression steel yields, obtain the value of a using:

$$a = \frac{A_s f_y - A_s' f_s'}{0.85 f_c' b} \quad \text{……………………………………………… (2.13)}$$

4. Obtain the nominal moment strength:

$$M_n = 0.85 f_c' ab\left(d - \frac{a}{2}\right) + A_s' f_y (d - d') \quad \text{.....................} (2.14)$$

$$M_u = \phi M_n \quad \text{.....................} (2.15)$$

5. When the compression steel does not yield:

   C = T..(2.16)

   $C_c + C_s = T_s$.. (2.17)

   $0.85f_c'ab + A_s'f_s' = A_sf_y$.. (2.18)

   where:

$$f_s' = 600\left(1 - \frac{d'}{c}\right) \quad \text{.....................} (2.19)$$

   or

$$f_s' = 600\left(1 - \beta_1 \frac{d'}{c}\right) \quad \text{.....................} (2.20)$$

   Equation (2.18) becomes:

$$0.85 f_c'(\beta_1 c)b + A_s' 600\left(1 - \frac{d'}{c}\right) = A_s f_y \quad \text{.....................} (2.21)$$

   In the simple form:

$$c = \frac{-B + \sqrt{B^2 + 4AC}}{2A} \quad \text{.....................} (2.22)$$

   where:

   $A = 0.85f_c'\beta_1 b$

   $B = 600A_s' - A_sf_y$

   $C = 600A_s'd'$

   In which the equivalent block can be obtained using $a = \beta_1 c$

6. The nominal moment strength is obtained using:

$$M_n = 0.85 f_c' ab\left(d - \frac{a}{2}\right) + A_s' f_y (d - d') \quad \text{.....................} (2.23)$$

7. Check if the tension steel yield using:

$$\varepsilon_s = \frac{d-c}{c}\varepsilon_{cu}' \geq \varepsilon_y \quad (2.24)$$

2.1.3.2. The design of the shear reinforcement

Based on SNI 03-2847-2002 section 13.1(1), the design due to shear shall satisfy:

$Ø V_n \geq V_u$……………………………………………………………….. (2.25)

with:

V_u = factored shear force

V_n = the nominal shear strength, which calculated from:

$V_n = V_c + V_s$…………………………………………………………… (2.26)

with,

V_c is the nominal shear strength provided by the concrete.

V_s is the nominal shear strength provided by shear reinforcement

Concrete shear strength for structural component which is loaded by shear and flexure (SNI 03-2847-2002 section 13.3(1)):

$$V_c = \left(\sqrt{\frac{f_c'}{6}}\right) b_w d \quad (2.27)$$

Shear strength for the design of the shear reinforcement which perpendicular to the structure axial axis is as follow (SNI 03-2847-2002 section 13.5(6(2))):

$$V_s = \frac{A_v f_y d}{s} \quad (2.28)$$

The shear reinforcement shall satisfy section 13.5(4(3)) and section 13.5(6(9)):

$$V_s < \left(\frac{\sqrt{f_c'}}{3}\right) b_w d \quad (2.29)$$

$$V_s < \frac{2}{3}\sqrt{f_c'}\, b_w d \quad (2.30)$$

with:

A_v = the area of the shear reinforcement

s = center to center spacing of shear reinforcement

V_s= shear strength

2.1.3.3. The design of the torsion reinforcement

The torsion effect can be neglected if: (SNI 03-2847-2002 section 13.6(1a))

$$T_u < \frac{\phi\sqrt{f_c'}}{12}\left(\frac{A_{cp}^{\ 2}}{P_{cp}}\right) \quad (2.31)$$

with:

T_u = torsion moment due to factored load

Ø = torsion reduction factor

P_{cp} = concrete perimeter

A_{cp} = area bordered by the concrete perimeter

The cross section dimension shall be able to resist torsion flexure strength: (SNI 03-2847-2002 section 13.6(3))

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^{\ 2}}\right)^2} \leq \phi\left(\frac{V_c}{b_w d} + \frac{2\sqrt{f_c'}}{3}\right) \quad (2.32)$$

with:

P_h = perimeter from the center of the outer shear reinforcement

A_{oh} = area within the line of the center of the outer shear reinforcement

2.1.3.4. The design of SMRF

SMRF members shall satisfy: (SNI 03-2847-2002 section 23.3(1))

1. $P_u < 0.1A_g f_c$'
2. Clear span of member $l_n > 4d$
3. Ratio $b_w/h > 0.3$
4. b_w >250mm
5. b_w < width of supporting member + ¾d

2.1.4. Column Design

The basic problem in column design is to establish the proportion of a reinforced concrete cross section whose theoretical strength, multiplied by a reduction factor, is just adequate to support the axial load and maximum moment in the column produced by factored design loads. This criterion can be summarized as:

$$P_u \leq \Phi P_n \text{ and } M_u \leq \Phi M_n \quad (2.42)$$

where P_u and M_u are the axial load and moment produced by factored load, and P_n and M_n are the theoretical axial and bending strength also referred as the nominal strength.

2.1.4.1. The design of the longitudinal reinforcement

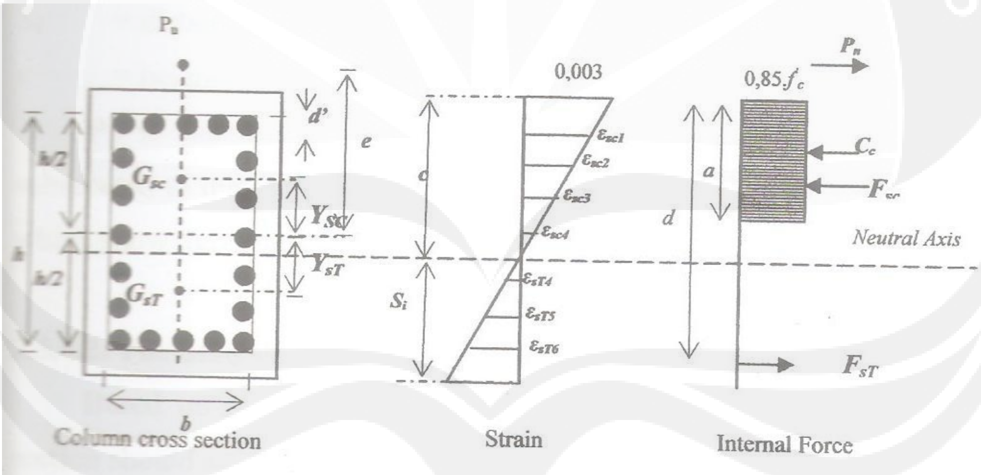

Figure 2.2 Columns with Bars at Four Faces

G_{SC} = Center of gravity of the steel compression force

G_{ST} = Center of gravity of the steel tension force

F_{SC} = Resultant of the steel compression force = $\sum A_s' f_{SC}$

F_{ST} = Resultant of the steel tension force = $\sum A_s f_{ST}$

f_{ST} = Steel tension stress

f_{SC} = Steel compression stress

The axial load capacity:

$$P_n = (0.85 f_c' b \beta_1 c) + F_{SC} + F_{ST} \quad (2.33)$$

The balance of the internal and external moment (about the centroid of column) shown in the equation:

$$M_n = P_n e = 0.85 f_c' b \beta_1 c \left(\frac{h}{2} - \frac{1}{2} \beta_1 c \right) + F_{SC} Y_{SC} + F_{ST} Y_{ST} \quad (2.34)$$

with:

F_{SC} = resultant of the compression force = $\sum A_s' f_{SC}$

F_{ST} = resultant of the tension force = $\sum A_s f_{ST}$

Trial and adjustment usually used for column connection with first assume the value of c, and then calculate the value of a. Stress of the reinforcement shall be calculated using:

$$F_{Si} = E_S \varepsilon_{Si} = E_s \varepsilon_c \left(\frac{S_i}{c} \right) = 600 \left(\frac{S_i}{c} \right) \quad (2.35)$$

with:

S_i= distance between center of the reinforcement to the neutral axis

c = height of the neutral axis from the edge of the outer compression strain

P_n is calculated based on the value of assumed c. The value of P_n is substituted to the M_n equation to get the value of c. Interaction diagram of P-M which shown the column capacity could be drawn.

2.1.4.2. The design of the shear reinforcement

The design of shear cross section should satisfy: (SNI 03-2847-2002 section 13.1(1))

$$\phi V_n \geq V_u \quad (2.36)$$

where:

V_n is the nominal shear force, from:

$$V_n = V_c + V_s \quad (2.37)$$

with V_c is the concrete shear force.

Concrete shear force for structural component which loaded with axial force shall satisfy: (SNI 03-2847-2002 section 13.3(2))

$$V_c = \left(1 + \frac{N_u}{14A_g}\right)\left(\frac{\sqrt{f_c'}}{6}\right)b_w d \quad \text{.......} \quad (2.38)$$

2.1.4.3. The design of the SMRF

According to SNI 03-2847-2002 section 23.4(2) the column flexure strength for SMRF shall satisfy:

$$\Sigma M_e \geq \left(\frac{6}{5}\right)\Sigma M_g \quad \text{.......} \quad (2.39)$$

with:

ΣM_e is the sum of the moment at the center of beam column connection, related to the column nominal flexure strength. Column flexure strength should be calculated for factorized axial force, based on the lateral forces direction, which result in the smallest value of flexure strength.

ΣM_g is the sum of moment at the center of beam column connection, related to the beam-beam nominal flexure strength. For the T-beam construction, which the slab is pulled to the face of the column, slab reinforcement at the effective width shall be calculated.

The column shear force for SMRF shall satisfy:

$$V_e = \frac{M_{pr1} + M_{pr2}}{h} \quad \text{.......} \quad (2.40)$$

with:

V_e = shear force

M_{pr1} = probable flexure strength at the top end of the column

M_{pr2} = probable flexure strength at the bottom end of column

h = clear height of the column

2.2. Building Performance Level

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building.

ATC-40 describes standard performance levels for structural performance as:

a. **Operational**
b. **Immediate Occupancy (IO)**: very limited structural damage has occurred. The risk of life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.
c. **Damage Control**: a range of IO and Life Safety (LS). It limits the structural damage beyond the Life Safety level, but occupancy is not the issue. E.g. the protection of significant architectural features of historic buildings or valuable contents
d. **Life Safety (LS)**: the injuries during the earthquake may occur; the risk of life-threatening injury from structural damage is very low.
e. **Limited Safety**: a range of LS and Collapse Prevention (CP), Structural Stability. Some critical structural deficiencies are mitigated.
f. **Structural Stability or Collapse Prevention (CP)**: Substantial damage to the structure has occurred, including stiffness and strength of the lateral force resisting system. However, all significant components o f the gravity load resisting system continue to carry their gravity demands.
g. **Not considered**

The performance level of a building is determined based upon its function and importance. Public building is expected to have a performance level of operational or immediate occupancy. A residential building must have a performance level of damage control or life safety. For the temporary structure is under the structural stability or not considered. The force deformation relationship as well as the structural performance levels is given in Figure 2.3.

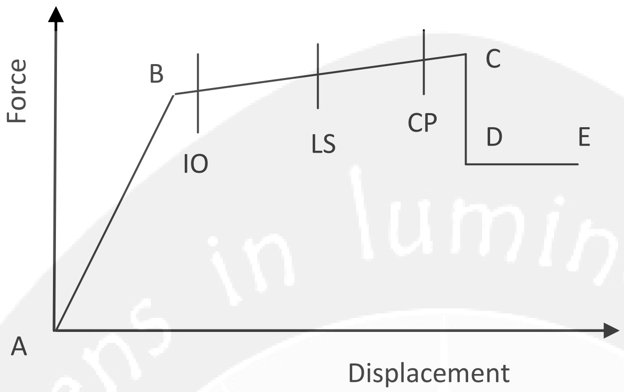

Figure 2.3 Force-deformation curve

Where: A = the origin
B = yielding
IO = immediate occupancy
LS = life safety
CP = collapse prevention
C = ultimate capacity
D = residual strength
E = total failure

Five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge. Three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge.

2.3. Nonlinear Static Pushover Analysis

Analysis methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis. In these the first two is suitable only when the structural loads are small and at no point the load will reach to collapse load. During earthquake loads the structural loading will reach to collapse load and the material stresses will be above yield stresses. So in this case material nonlinearity and geometrical nonlinearity should be incorporated into the analysis to get better result.

Nonlinear static pushover analysis or Push-over analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., parabolic, triangular or uniform). The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various structural components is recorded. In the structural design process a series of iterations are usually required during which, the structural deficiencies observed in iteration is rectified and followed by another. This iterative analysis and design procedure continues until the design satisfies pre-established performance criteria. In the other hand, static pushover analysis evaluates the real strength of the structure so that it will be useful and effective for performance based design.

This method is considered as a step forward from the use of linear analysis, because they are based on a more accurate estimate of the distributed yielding within a structure, rather than an assumed, uniform ductility. The generation of the pushover curve also provides the nonlinear behavior of a structure under lateral load. However, it is important to remember that pushover methods have no rigorous theoretical basis, and may be inaccurate if the assumed load distribution is incorrect.

For example, the use of a load pattern based on the fundamental mode shape may be inaccurate if higher modes are significant, and the use of any fixed load pattern may be unrealistic if yielding is not uniformly distributed, so that the stiffness profile changes as the structural yields.

This analysis provides data on the strength and ductility of the structure which otherwise cannot be predicted. Base shear *versus* top displacement curve of the structure, called pushover curves, are essential outcomes of pushover analysis. These curves are useful in ascertaining whether a structure is capable of sustaining certain level of seismic load.

The basic steps of POA are:

1. Assume the nonlinear force-displacement relationship of individual elements of structure (including yield strength, post yield stiffness and stiffness degradation, etc)
2. Calculate the target displacement of structure
3. Select a reasonable lateral load pattern, and pushing the structure under this load pattern which is monotonically increasing step by step, when a structural member yields, then its stiffness is modified, until the roof displacement of structure is up to the target displacement or the structure collapses. At this time, the evaluation of seismic performance of structure is obtained.

The main output of a pushover analysis is in term of response demand versus capacity as shown in Figure 2.4. Demand and capacity are mutually dependent. As displacements increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Inelastic displacements increase damping and reduce demand. The capacity spectrum method reduces demand to find an intersection with the capacity spectrum where the displacement is consistent with the implied damping. The intersection between capacity and demand curve develop the performance point. At the performance point, capacity and demand are equal. The displacement of the performance point is the target displacement.

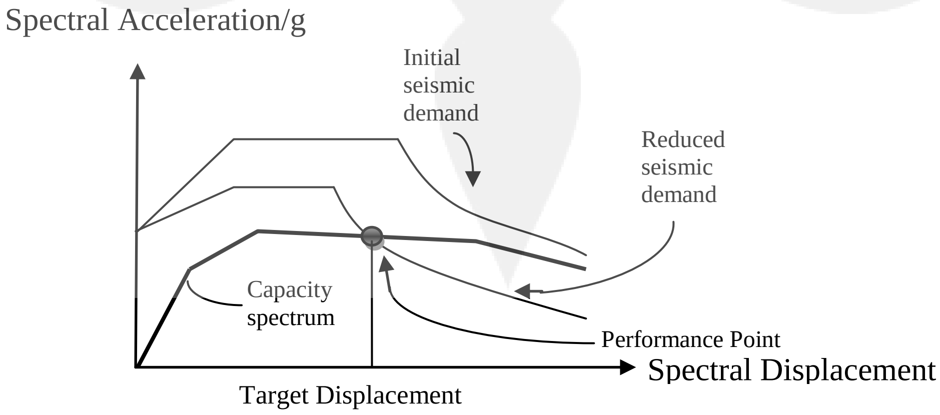

Figure 2.4 Demand vs Capacity Curve

2.4. Target Displacement and Lateral Load Pattern

The target displacement and lateral load pattern is very important for POA to evaluate the seismic performance of structures. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The load patterns are intended to represent and bound the distribution of inertia forces in a design earthquake.

One procedure for evaluating the target displacement as per FEMA 273 is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad (2.39)$$

where:

T_e = Effective fundamental period of the building in the direction under consideration, sec

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (2.40)$$

T_i =Elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis

K_i =Elastic lateral stiffness of the building in the direction under consideration

K_e =Effective lateral stiffness of the building in the direction under consideration

C_0 =Modification factor to relate spectral displacement and likely building roof displacement.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

C_2 =Modification factor to represent the effect hysteresis shape on the maximum displacement response. The value can be seen in Table3-1 FEMA 273.

C_3 =Modification factor to represent increased displacements due to dynamic P-Δ effect

S_a =Response spectrum acceleration, at the effective fundamental period and dumping ratio of the building in the direction under consideration

Based on SNI 03-1726-2002, nominal earthquake loads in the form of base shear load on the building structure can be calculated by equivalent static analysis approach. Nominal base shear load in this equation is calculated with equation (2.7)

$$V_n = \frac{C_1 I}{R} W_t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

The nominal base shear, V, should be distributed along the height structure as the lateral load, Fi, which works on the center of mass of the i-th floor. The lateral load F_i applied at any floor level i shall be determined from the following equation

$$F_i = C_{VX} V \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.41)$$

where:

C_{VX} = Vertical distribution factor

V = Pseudo lateral load from equation (2.7)

The vertical distribution factor patterns according to FEMA 273 are:

1. Uniform distribution

   The uniform distribution can be calculated by the equation

   $C_{VX} = \dfrac{m_i}{\sum m_j}$, where m_i is the storey mass

2. Equivalent lateral force distribution

   $$C_{VX} = \frac{m_i h_i^{\,k}}{\Sigma(m_j h_j^{\,k})}$$

   Where:

   m_i = story mass of level i

   h_i = height from the base of a building to floor level i

   k = can be computed by

$$k = \begin{cases} 1.0; \; for T \le 0.5 \\ 1.0 + \dfrac{2.5 - 0.5}{T - 0.5}; \; for 0.5 < T < 2.5 \\ 2.0; \; for T \ge 2.5 \end{cases}$$

3. SRSS distribution

When the periods and modes of a structure is known, the story shear forces can be calculated using SRSS (Square Root of the Sum of the Square), then the equivalent lateral load can also be obtained, and it can be regarded as the lateral load pattern for the next step.

Define F_{ij}, Q_{ij} as lateral load, story shear of floor *i* corresponding to *j*th mode, Q_i as story shear of floor *i* using SRSS of *N* modes, P_i as equivalent lateral load of floor *i*, then pattern (3) is described as following:

$$F_{ij} = \alpha_j \gamma_j X_{ij} W_i$$

$$Q_{ij} = \sum_{m=i}^{n} F_{mj}$$

$$Q_i = \sqrt{\sum_{j=1}^{N} Q_{ij}}$$

$$P_i = Q_i - Q_{i+1}$$

Where α_j is horizon seismic effect coefficient corresponding to the natural period of *j*th mode of the structure. X_{ij} is relative horizon displacement of floor *i* corresponding to *j*th mode, *N* is number of modes, *n* is number of stories, W_i is weight of floor *i*.