Title: Outrigger System Study for Tall Building Structure With Central Core and Square Floor Plate

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Outrigger System Study for Tall Building Structure  
With Central Core and Square Floor Plate

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1. Introduction

Outrigger braced tall building structure usually consists of a stiff central core, connected to the exterior columns by flexural stiff cantilevers at the outrigger floors and floor members (slab and floor beam) at the typical floors. The outrigger system has been widely used on tall buildings, the system effectively create a mechanism engaging the full depth of the building frame in resisting lateral loadings.

Consider a tall building structure with a square floor plate and a central core. When lateral loadings act on the building structure, the outriggers, which connected the central core and the perimeter tower columns, resist the rotation of the central core, causing the lateral deflections and moments in the core to be smaller than if the freestanding core alone resisted the loading. The result is to increase the effective depth of the building structure, beyond those provided by the central core, by inducing tension and compression in the perimeter columns.

Many studies demonstrated the behavior and effectiveness of the outrigger system, but without considering the stiffness of the floor members, which link between the central core and the perimeter columns at the typical tower floors.

This paper is to demonstrate the effectiveness of the outrigger system for a central cored building structure with two outrigger floors and perimeter columns, and also considers the slab stiffness at the typical floors.

2. Study Model 1

(1) Model Description

A sixty-story concrete tower is being used as our study model. The tower has a square floor plate of 34.5 meters x 34.5 meters. There is a 16 meters x 16 meters concrete central core and twenty (20) perimeter concrete columns. On the typical tower floors, 250mm two-way flat slab spans between the central core and the perimeter columns. There are no floor beams on the typical tower floors. The typical floor-to-floor height is 3.0 meters.

There are two outrigger floors for the tower. The lower outrigger floor is located at the lower pit floor, where the mechanical plant rooms are located. The lower pit floor is 7.0 meters in height. The upper outrigger floor is located between the tower typical floor and the penthouse floor, and is at level 56.

Both the lower and upper outrigger systems consist of outriggers connecting the central core to the perimeter columns and a belt wall system around the building perimeter on the outrigger floors. The combination of the outriggers connecting the core to the perimeter columns and the belt wall created a stiff outrigger system.
Refer to Figure B-1 for isometric view of the tower framing.

Figure B-1 – Isometric View of Study Model

(2) Simplified Analytical Model Without Considering Floor Slab Stiffness

The following simplified analytical model is based on the paper “Tall Building Structures, Analysis and Design” by Bryan Stafford Smith, Alex Coull. It is based on the following assumptions:

A. The structure is linearly elastic.
B. Only axial forces are induced in the perimeter columns.
C. The outriggers are rigidly attached to the core and the core is rigidly attached to the foundation.
D. The sectional properties of the core, column and outriggers are uniform throughout the height.
E. The stiffness provided by the typical floor slab connecting the core and the perimeter columns are ignored.
Analyses for a two-outrigger structure subjected to a uniform distributed load (CASE 1) and a triangular distributed load with its apex at the base (CASE 2) are presented below.

Case 1: Uniform load distribution

Starting from the statically determinate freestanding core, a two-outrigger structure is a twice-redundant system, and two compatibility equations are needed. The compatibility equations state the equivalence of the rotation of the core to the rotation of the outrigger at the two outrigger levels. The rotation of the core is expressed in terms of its bending deformation, and that of the outrigger in terms of the axial deformation of the column and the bending of the outrigger.

Analysis of the two-outrigger structure subjected to a uniformly distributed horizontal load is shown in Fig. B-2. The bending moment diagram for the core consists of the moment diagram of the cantilever subjected to uniform load reduced by the outrigger restraining moments. From the moment-area method, the core rotation at x1 and x2 are:

\[ \theta_1 = \frac{1}{EI} \int_{x_1}^{x_2} \left( \frac{wx^2}{2} - M_1 \right) dx + \frac{1}{EI} \int_{x_2}^{H} \left( \frac{wx^2}{2} - M_1 - M_2 \right) dx \quad \text{.....(C1.1)} \]

And

\[ \theta_2 = \frac{1}{EI} \int_{x_2}^{H} \left( \frac{wx^2}{2} - M_1 - M_2 \right) dx \quad \text{.....(C1.2)} \]

Where

- \( EI \): flexural rigidity of the core
- \( H \): total height of the core
- \( w \): intensity of horizontal loading
- \( x_1, x_2 \): respective heights of outriggers 1 and 2 from the top of the core
- \( M_1, M_2 \): respective restraining moments on the core

The rotation of the outriggers at the inboard ends consists two components. The first component is due to the differential axial deformation of the columns, and the second component is due to the outriggers bending under the action of the column forces at their outboard ends.

The rotation of the outrigger at x1 is:

\[ \theta_1 = \frac{2 M_1 (H - x_1)}{d^2 (EA)_c} + \frac{2 M_2 (H - x_2)}{d^2 (EA)_c} + \frac{M_1 d}{12 (EI)_o} \quad \text{.....(C1.3)} \]

And the rotation of the outrigger at x2 is:

\[ \theta_2 = \frac{2 (M_1 + M_2) (H - x_2)}{d^2 (EA)_c} + \frac{M_2 d}{12 (EI)_o} \quad \text{.....(C1.4)} \]

Where

- \((EA)_c\): axial rigidity of the column
- \(d/2\): horizontal distance of column from the centroid of the core
- \((EI)_o\): effective flexural rigidity of the outrigger

The effective flexural rigidity \((EI)_o\) of the outrigger, allowing for the wide-column effect of the core, can be expressed in terms of the actual outrigger flexural stiffness \((EI)'_o\) as:
\[(EI)_o = \left(1 + \frac{a}{b} \right)^3 \left(EI'\right)_o \] .....(C1.5)

Where \((EI')_o\) : actual flexural rigidity of the outrigger

\na : horizontal distance from the centroid of the core to edge

\nb : the net length of the outrigger

Equating the rotations of the core and outrigger at \(x_1\) and \(x_2\) provided the compatibility equations as follows:

\[
\frac{2M_1(H - x_1)}{d^2(EA)_c} + \frac{2M_2(H - x_2)}{d^2(EA)_c} + \frac{M_d}{12(EI)_o} = \frac{1}{EI} \left[ \int_{x_1}^{x_2} \left( \frac{wx^2}{2} - M_1 \right) dx + \int_{x_2}^{H} \left( \frac{wx^2}{2} - M_1 - M_2 \right) dx \right]
\]

\[
\frac{2(M_1 + M_2)(H - x_2)}{d^2(EA)_c} + \frac{M_d}{12(EI)_o} = \frac{1}{EI} \int_{x_2}^{H} \left( \frac{wx^2}{2} - M_1 - M_2 \right) dx
\]

Define \(S\) and \(S_1\) as:

\[
S = \frac{1}{EI} + \frac{2}{d^2(EA)_c}
\]

\[
S_1 = \frac{d}{12(EI)_o}
\]

Rewriting (C1.6) and (C1.7) as:

\[
M_1[S_1 + S(H - x_1)] + M_2S(H - x_2) = \frac{w}{6EI} \left( H^3 - x_i^3 \right)
\]

\[
M_1S(H - x_2) + M_2\left[ S_1 + S(H - x_2) \right] = \frac{w}{6EI} \left( H^3 - x_i^3 \right)
\]

The solution of (C1.10) and (C1.11) gives the restraining moment applied to the core by the outriggers at \(x_1, x_2\):

\[
M_1 = \frac{w}{6EI} \left[ \frac{S_1(H^3 - x_i^3) + S(H - x_2)(x_i^3 - x_i^3)}{S_i^2 + S_2(S(2H - x_i - x_2) + S^2(H - x_2)(x_2 - x_i))} \right]
\]

\[
M_2 = \frac{w}{6EI} \left[ \frac{S_2(H^3 - x_i^3) + S(H - x_1)(x_1^3 - x_i^3) - (H - x_2)(H^3 - x_2^3)}{S_1^2 + S_2(S(2H - x_i - x_2) + S^2(H - x_2)(x_2 - x_i))} \right]
\]

Having solved the outrigger restraining moments, the resulting moment in the core can be expressed as:

\[
M_x = \frac{wx^2}{2} - M_1 - M_2
\]

in which \(M_1\) is for \(x > x_1\), and \(M_2\) is for \(x > x_2\). The horizontal deflection equation of the structure may be complicated; the tower top drift can be expressed as:

\[
\Delta_0 = \frac{wH^4}{8EI} - \frac{1}{2EI} \left[ M_1(H^2 - x_i^2) - M_2(H^2 - x_2^2) \right]
\]
in which the first term on the right-hand side represents the drift at the top of the core acting as a free vertical cantilever subjected to uniform distributed load, while the two part of the second term represent the reductions in the top drift due to the outrigger restraining moments.

The preceding analysis allows an assessment of the optimum levels of the outriggers to minimize the horizontal tower top deflection by maximizing the second term on the right-hand side of (C1.15), which maximized by differentiating with respect first to $x_1$, then to $x_2$ thus:

\[
\frac{d\Delta_0}{dx_1} = \left(H^2 - x_1^2\right) \frac{dM_1}{dx_1} + \left(H^2 - x_2^2\right) \frac{dM_2}{dx_1} - 2x_1 M_1 = 0 \quad \ldots \text{(C1.16)}
\]

\[
\frac{d\Delta_0}{dx_2} = \left(H^2 - x_1^2\right) \frac{dM_1}{dx_2} + \left(H^2 - x_2^2\right) \frac{dM_2}{dx_2} - 2x_2 M_2 = 0 \quad \ldots \text{(C1.17)}
\]

Define:

\[
\alpha \equiv \frac{EI}{(EA) \left(d^2/2\right)} \quad \ldots \text{(C1.18)}
\]

\[
\beta \equiv \frac{EI \cdot d}{(EI) \omega H} \quad \ldots \text{(C1.19)}
\]

\[
\omega = \frac{\beta}{12(1+\alpha)} \quad \text{(i.e. } \frac{S}{S'} = \frac{1}{H\omega}) \quad \ldots \text{(C1.20)}
\]

(C1.16) and (C1.17) can be solved by numerical method for the values of $x_1/H$ and $x_2/H$ expressed in term of $\omega$.

\[
x_1 = F_1(\omega)
\]

\[
= 0.479\omega^6 - 1.731\omega^5 + 2.551\omega^4 - 2.028\omega^3 + 1.024\omega^2 - 0.446\omega + 0.312
\]

And

\[
x_2 = F_2(\omega)
\]

\[
= 1.817\omega^6 - 6.489\omega^5 + 9.329\omega^4 - 7.026\omega^3 + 3.140\omega^2 - 1.080\omega + 0.685
\]

The parameter $\omega$, which is nondimensional, is the characteristic structural parameter for a uniform structure with flexible outriggers.

**Case 2: Triangular load distribution**

Analysis of the two-outrigger structure subjected to a triangular distributed horizontal load with its apex at the base as shown in Fig. B-3 follows the same procedure as described for two-outrigger structure subjected to uniformly distributed loading.

From the moment-area method, the core rotation at $x_3$ and $x_4$ are as follows:

![Figure B-3](image-url)
\[
\theta_3 = \frac{1}{EI} \int_{x_3}^{x_4} \left( \frac{wx^2}{2} \left(1 - \frac{x}{3H}\right) - M_3 \right) dx + \frac{1}{EI} \int_{x_3}^{x_4} \left( \frac{wx^2}{2} \left(1 - \frac{x}{3H}\right) - M_3 - M_4 \right) dx \quad \cdots \text{(C1.23)}
\]

\[
\theta_4 = \frac{1}{EI} \int_{x_4}^{x_4} \left( \frac{wx^2}{2} \left(1 - \frac{x}{3H}\right) - M_3 - M_4 \right) dx \quad \cdots \text{(C1.24)}
\]

Where
- \( EI \): flexural rigidity of the core
- \( H \): total height of the core
- \( w \): intensity of horizontal loading
- \( x_3, x_4 \): respective heights of outriggers 3 and 4 from the top of the core
- \( M_3, M_4 \): respective restraining moments on the core

The rotation of the outrigger at \( x_3 \) is:
\[
\theta_3 = \frac{2M_3(H - x_3)}{d^2(EA)_c} + \frac{2M_4(H - x_4)}{d^2(EA)_c} + \frac{M_4d}{12(EI)_o} \quad \cdots \text{(C1.25)}
\]

And at \( x_4 \):
\[
\theta_4 = \frac{2(M_3 + M_4)(H - x_4)}{d^2(EA)_c} + \frac{M_4d}{12(EI)_o} \quad \cdots \text{(C1.26)}
\]

Where
- \((EA)_c\): axial rigidity of the column
- \(d/2\): horizontal distance from the centroid of the core
- \((EI)_o\): effective flexural rigidity of the outrigger

Equating the rotations of the core and outrigger at \( x_3 \) and \( x_4 \), provided the compatibility equations:
\[
\frac{2M_3(H - x_3)}{d^2(EA)_c} + \frac{2M_4(H - x_4)}{d^2(EA)_c} + \frac{M_4d}{12(EI)_o} = \frac{1}{EI} \int_{x_3}^{x_4} \left( \frac{wx^2}{2} \left(1 - \frac{x}{3H}\right) - M_3 - M_4 \right) dx \quad \cdots \text{(C1.27)}
\]

Rewriting (C1.27) and (C1.28)
\[
M_3[S_1 + S(H - x_3)] + M_4S(H - x_4) = \frac{w}{24EI} \left(4Hx_3^3 - x_3^4 - 3H^4\right) \quad \cdots \text{(C1.29)}
\]

\[
M_3S(H - x_3) + M_4[S_1 + S(H - x_4)] = \frac{w}{24EI} \left(4Hx_4^3 - x_4^4 - 3H^4\right) \quad \cdots \text{(C1.30)}
\]

in which \( S \) and \( S_1 \) are defined in (C1.8) and (C1.9). The simultaneous solution of (C1.29) and (C1.30) gives the restraining moment applied to the core by the outrigger at \( x_3, x_4 \),
\[
M_i = \frac{w}{24EI} \left[ \frac{3H^4S_1 + S_1x_1^3 + 4H^3S(x_1^3 - x_1^4) + S_1(x_4^4 - x_1^4) + H[S(x_3^4 + 4x_1^4x_4 - 5x_1^4 - 4Sx_1^4)]}{(S^2(H - x_3)^2 - (HS + S_1 - Sx_3)(HS + S_1 - Sx_4))} \right] \quad \cdots \text{(C1.31)}
\]
\[ M_4 = \frac{w}{24EI} \left[ \left( S(H-x_4)\left( 3H^4-4Hx_4^3+x_4^4 \right) - (HS+S_1-Sx_4)H^3 \right) \right. \]
\[ \left. \left( S^2(H-x_4)^2-(HS+S_1-Sx_4)(HS+S_1-Sx_4) \right) \right] \]  
...(C1.32)

The resulting moment in the core can be expressed generally by
\[ M_x = \frac{wx^2}{3} - M_3 - M_4 \]  
...(C1.33)
in which \( M_3 \) is for \( x>x_3 \), and \( M_4 \) is for \( x>x_4 \). The tower top drift can be expressed as:
\[ \Delta_0 = \frac{11wH^4}{120EI} - \frac{1}{2EI} \left[ M_3(H^2-x_3^2) - M_4(H^2-x_4^2) \right] \]  
...(C1.34)
in which the first term on the right-hand side represents the drift at top of the core acting as a free vertical cantilever subjected to the triangular distribution load with its apex at the base, while the two part of the second term represent the reductions in drift due to the outrigger restraining moments.

Differentiate the right-hand side of (C1.34) with respect first to \( x_3 \), then to \( x_4 \) for assessment of the optimum levels of the outriggers to minimize the horizontal top drift, thus
\[ \frac{d\Delta_0}{dx_3} = \left( H^2-x_3^2 \right) \frac{dM_3}{dx_3} + \left( H^2-x_4^2 \right) \frac{dM_4}{dx_3} - 2x_3M_3 = 0 \]  
...(C1.35)
\[ \frac{d\Delta_0}{dx_4} = \left( H^2-x_3^2 \right) \frac{dM_3}{dx_4} + \left( H^2-x_4^2 \right) \frac{dM_4}{dx_4} - 2x_4M_4 = 0 \]  
...(C1.36)

(C1.35) and (C1.36) can be solved simultaneously in numerical method for the values of \( x_3/H \) and \( x_4/H \) expressed in term of \( \omega \).

\[ \frac{x_3}{H} = F_3(\omega) \]  
...(C1.37)
\[ 0.509\omega^6 - 1.838\omega^5 + 2.694\omega^4 - 2.108\omega^3 + 1.026\omega^2 - 0.421\omega + 0.292 \]

And
\[ \frac{x_4}{H} = F_4(\omega) \]  
...(C1.38)
\[ 1.733\omega^6 - 6.201\omega^5 + 8.939\omega^4 - 6.764\omega^3 + 3.049\omega^2 - 1.059\omega + 0.663 \]

The characteristic structural non-dimensional parameter \( \omega \) is defined in (C1.20) Equation (C1.21), (C1.22), (C1.37) and (C1.38) may be used to find the optimum outrigger levels, \( x_3/H \) & \( x_2/H \) for uniformly distributed horizontal load, and \( x_3/H \& x_4/H \) for triangular distributed horizontal load with its apex at the base.
The above-mentioned optimum outrigger locations in terms of $\omega$ are plotted graphically in Fig B-4.

The conclusion is that using the compatibility method and *without considering the typical floor slab stiffness*, the analysis of a two-outrigger structure subjected to uniformly distributed loading and a triangular distributed horizontal loading with its apex at the base have almost identical location of optimum outrigger floor.

*For the 60 stories study model and assume a relatively rigid outrigger system and without considering the typical floor slab stiffness, the most optimum location of the upper outrigger system to minimize the tower top displacement is located at approximately level 42, and the most optimum location of the lower outrigger system is located at approximately level 20.*

This simplified analytical study does not address story drift, which is one of the criteria in the design of tall building structure subject to lateral loading.

### (3) Study Model Considering Typical Floor Slab Stiffness

In order to compare the result of the simplified analytical model stated in section B (2) to the result of the three-dimensional structural analytical model, which considers floor slab and perimeter column stiffnesses in axial, shear and flexural, a three-dimensional Etabs model for the study model was established.

The primary lateral stiffness of the study model consists of the central concrete core linked to the perimeter concrete columns by two outrigger floors and the typical tower floor slab. The upper outrigger floor is fixed at level 56 (between the typical tower floor and the upper penthouse floor), the lower outrigger system moves between levels 20 to level 35 in order to find the most optimum location for minimum roof top displacement.

The two outrigger floors consist of concrete outrigger connecting the core and the perimeter columns and a belt wall system around the outrigger floor perimeter. The belt wall system utilizes the 300mm thick concrete slab above and below the outrigger floors to transfer tension and compression forces, forming a couple between the central core and the perimeter columns.

The design wind loading for the study model is shown below in Figure B-5.
The above-mentioned study model also includes the stiffness of the typical tower floor slab, which was modeled as a shell element. 50% I gross of the slab was used in the analysis considering the cracked section properties of the floor slab under gravity loadings.

The following figures (B-6 and B-7) demonstrate the displacement and story drift of the study model subjected to design wind loading with different location of the lower outrigger system (upper outrigger system is located at level 56) while considering the typical floor slab’s stiffness and modeling them as a shell elements.

**Figure B-6**

![Diagram](image_url)
Figures B-6 and B-7 concluded that while considering the typical tower floor slab as part of the link between the central core and the perimeter columns, the location of the lower outrigger floor (to be located between level 23 to 33) do not have significant impact on the tower top displacement, but will change the story drift pattern. Minimum story drift is shown for the lower outrigger floor located at between levels 31 to 33.

Using the similar analytical model as stated above except changing the connecting slab from shell elements to beam elements. These beam elements for the slab have a member depth equals to the slab thickness, an effective member width of 12 times the slab thickness or half the distance to the adjacent “beam”. The displacement and story drift of the study model with different location of the lower outrigger system (upper outrigger system is located at level 56) are in Figures B-8 and B-9.
Figures B-8 and B-9 concluded that:

1. The contribution of stiffness by the tower typical floor slab is important;
2. Considering the slab stiffness through the use of the beam element (one way beam flexural) makes the overall system more flexible as compared to fully utilize the two-way slab stiffness (modeled as shell elements). The outcome is that the overall system is almost as flexible as the system without considering the slab stiffness (see Fig. B10 and B11);
3. The location of the lower outrigger floor (to be located between level 23 to 33) does not have
significant impact on the roof top displacement. The most optimum lower outrigger location to minimize tower top displacement is located at level 23.

4. The most optimum lower outrigger location to minimize story drift is located at level 27.

Using the similar analytical model as stated above except deleting completely the typical floor slab stiffness, the displacement and story drift of the study model with different location of the lower outrigger system (upper outrigger system is located at level 56) are shown in Figure B10 and B11.
Combining the data from Figures B-6, B-8 and B-10 on Figure B-12, we can conclude that:

1. Having the outrigger system (both upper and lower outriggers) is effective in controlling lateral displacement and drift regardless of the outrigger location and the way the typical floor slab’s stiffness is modeled.

2. For a two-outrigger system, the upper outrigger is to be located at (2/3 X total height of the tower). Relocating the outrigger in within (1/6 X total height of the tower) zone will not significantly change the tower top displacement, but will alter the magnitude of the story drift.

3. For a two-outrigger system, the lower outrigger is to be located at 1/3 X total height of the tower. Relocating the outrigger in within (1/6 X total height of the tower) zone will not significantly change the tower top displacement, but will alter the magnitude of the story drift.

4. Modeling the typical floor slab as shell element will capture the correct slab stiffness contribution as link between the central core and the perimeter columns.

5. Modeling the typical floor slab as beam element will make the overall system perform as without any slab link.

6. Removing the lower outrigger system will increase the tower top displacement by 3.0 times.

7. Removing both the upper and lower outrigger systems and the typical floor slab link will make the tower deflect as a cantilever, having only the central core provide all the lateral stiffness. This provides a check to the analytical model used.

![Figure B-12](image)

**3. Study Model 2 For Contribution of Slab Stiffness**

In order to further study the contribution of the typical floor slab stiffness to the overall outrigger system behavior, a second study model is created.

**(1) Model Description**

A sixty-story concrete tower and a forty-story concrete tower are being used as our study models. Both towers have a square floor plate and a 16 meters x 16 meters concrete central core. On the typical tower floors, two-way flat slab spans between the central core and the perimeter columns.
The typical floor-to-floor height is 3.0 meters. For both towers, the clear distance between the central core and the perimeter columns varies between seven (7) to nine (9) meters with linking slab thicknesses of 250mm and 320mm respectively. For both towers, there are two outrigger floors. The top outrigger floors are located at the tower top, the lower outrigger location moves as we intended to find the most optimum location for minimum roof displacement. Refer to Figure C-1 for tower perimeters.

(2) Study Model With The Roof Top Bell wall

Figures C-2 and C-3 indicate the relationship of the ratio of the tower top displacement with the lower tower outrigger (indicated as percentage of roof displacement ratio without the lower outrigger) vs. the location of the second outrigger (indicated as percentage of tower height). For both towers, two clear distances between the central core and the perimeter columns are used; these clear distances are to associate with slab thickness of 250mm and 320mm respectively. This is to reflect the changes in the typical floor slab stiffness.
(3) Study Model Without The Roof Top Bell wall

Figures C-4 and C-5 indicate the relationship of the ratio of the tower top displacement with the lower tower outrigger (indicated as percentage of roof displacement ratio without the lower outrigger) vs. the location of the second outrigger (indicated as percentage of tower height). For both towers, two clear distances between the central core and the perimeter columns are used; these clear distances are to associate with slab thickness of 250mm and 320mm respectively. This is to reflect the change in the typical floor slab stiffness.
4. Summary

We can conclude from the above studies that:

1. The outrigger system is efficient in increasing overall lateral stiffness;
2. The contribution of the slab stiffness on the total lateral tower stiffness is important and it should be considered;
3. For taller towers, the contribution of the slab stiffness is more significant;
4. Using shell elements in modeling the connecting floor slab will capture more contribution from the slab than modeling the slab as equivalent beam with effective width;
5. The optimum location of the lower outrigger floor is approximately at mid-height between the base of the tower and the upper outrigger floor.
6. Roof top bell-wall system will contribute to the overall tower stiffness and lessen the effect of the tower floor slab contribution. In other words, for a system with roof top bell-wall, the typical floor slab ‘s stiffness contribution is less important;
7. For taller towers, the effectiveness of the outrigger system is more significant;
8. Optimizing for roof top displacement and inter-story drift are different issues.

References