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Structural Effect on Columns of Existing Low-Rise Buildings due to Installation of Rooftop Towers - A Case Study

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Abstract: Telecommunication towers are vital components of any telecommunication network, as the last-mile connectivity of any wireless network is usually provided by these towers. The amount of expenditure, regulatory constraint and construction related challenges for a construction of a green field tower is usually high, hence telecommunication operators started to construct towers as an alternative for greenfield towers, especially in urban areas. However, ensuring the structural stability and integrity of the host structure is essential when installing rooftop towers, as almost all such buildings have not been initially designed considering additional loads from a roof top tower. In case of occurrence of a major structural issue of a building after installing a rooftop tower would lose the advantage of rooftop towers over green field towers as a sustainable alternative. Very limited scientific research on rooftop towers and the effects of rooftop tower installation on existing structures have been carried out in Sri Lanka as well as in global context. Accordingly, this study was undertaken to study structural effects on columns of existing low-rise buildings due to installation of a rooftop tower through a case study selected based on a field survey of buildings having rooftop towers. By Computer Aided structural modelling of the selected building while changing the location of the rooftop tower on the rooftop, effects of physical location of tower on structural performance of relevant columns were studied. According to this study, it was found that the tower location of a rooftop tower is highly influential on additional stresses in columns and would exceed the design capacities of columns especially when the tower is located close to a corner or edge of a rectangular building. Further, relevant to the selected case study, it was found that cracks reported in columns in this particular case would be most likely due the overstressing of columns, and immediate retrofitting and strengthening of defective columns would be required.

Keywords: Telecommunication tower, Rooftop tower, Host structure

1. Introduction

Telecommunication has become an essential need in the present society as a lot of day-to-day activities have become entangled with telecommunication services. Mobile banking, e-learning, e-commerce, and various social media platforms like Facebook, WhatsApp, etc. are a few of the services that contribute to the essential nature of telecommunication services in the modern society.

As per the latest data from Telecommunication Regulatory Commission of Sri Lanka - TRCSL [1], the total number of cellular mobile telephone subscribers are about 28 million, and the total number of fixed-line telephone subscribers are more than 2.5 million, in Sri Lanka. Most of these subscribers expect uninterrupted, high-quality telecommunication service in this context. Therefore, all telecommunication operators try to provide such services to retain their present customers and attract new customers for their business continuity.

As a majority of subscribers are mobile subscribers as highlighted in TRCSL data, telecommunication towers are essential to maintain the last-mile connectivity between subscribers and core networks of operators. However, construction of telecommunication towers, especially in urban and suburban areas, is a challenging task due to implications like scarcity of land, high construction costs, regulatory restrictions, social and cultural concerns, etc. As an alternative solution to these concerns, telecommunication operators started

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to construct relatively shorter towers on rooftops of existing buildings as the height of a building itself would beneficially add on to the total effective height of the tower. However, ensuring the structural stability of host structures is essential as rooftop towers may transfer considerable excessive stresses on structural elements of host structures.

Presently, TRCSL has introduced regulations to obtain structural certification for host structures prior to the installation of a tower, considering the above-mentioned structural effects of tower installation. Nevertheless, effective implementation of this rule is questionable as the availability of structural drawings in most existing structures is rare in local context (especially relevant to domestic and commercial structures owned by individuals). Therefore, only an approximate assessment subjected to a considerable number of assumptions would be performed in this type of structure, and the accuracy of such an analysis would be doubtful. Further, there are a considerable number of rooftop towers in the country that were constructed prior to the implementation of this rule.

Failures of rooftop towers causing damages to host structures have been reported especially in India in certain occasions. Figure 1 shows a failure of a rooftop tower causing damages to the host structure during cyclone Phailin in 2013 [2].

Figure 1 - Failure of a Tower in Chennai in India during Cyclone Phailin in 2013 [2]

Therefore, this study focused on investigating structural effects on columns due to the installation of rooftop towers through a case study of a selected building, which was selected after a field survey.

A very limited number of publications are available, especially in the local context, relevant to rooftop tower construction. A study done by Gunathilaka [3] recommended that installation of a tower in the middle of a slab panel with appropriate beam system would reduce the additional column loads on the relevant panel due to tower installation. Other than this study, no other local studies focused on structural effects on host structures by roof to towers was found.

Aseem and Quadir [4] had studied effects of a rooftop mounted telecommunication tower on the design of a building. According to findings of their study, it is reported that column loads of host structures have been increased by a considerable amount due to a rooftop tower installation. Hence, importance of proper design check of the structural members of an existing structure before installation of a telecommunication tower has been highlighted.

Malviya & Jamle [5] studied seismic performance of a building with a roof top tower. According to results of this study, the location of tower on the rooftop of the building greatly affects the seismic performance of the building, and a location close to centre of the building was found to be the best location under seismic loading.

A study carried out by Vikaskumar et al. [6] on influence of telecommunication tower on host structure highlighted that forces in members of host structure considerably increase with the effect of the tower on rooftop under wind and seismic loading and amount of increase depend on the location of the tower on the rooftop. Hence, restrengthening requirements of certain members were also highlighted.

However, none of these studies focused on detailed study of additional stresses developed in columns due to additional forces from rooftop towers, even though possibility of increasing member forces in host structures was discussed. Therefore, this study focused on studying structural effects on columns of existing low-rise buildings due to installation of rooftop towers and necessary actions that should be taken to minimise such effects through a case study selected based on a field survey of buildings with rooftop towers.

2. Methodology

There are different structural systems available for rooftop towers. The following are the common structural systems used in Sri Lanka:

1. Self-supporting lattice towers
2. Monopole
3. Guy masts
4. Poles with supporting struts

Usually, if the antenna load on the proposed tower is large and the required height above the roof level is high, self-supporting lattice towers would be the preferred as the structural system for such rooftop towers.

Therefore, a considerably high percentage of rooftop towers presently in the country belong to the self-supporting lattice tower category. Also, the additional stresses that would transfer to host structures are high when self-supporting lattice towers are installed on roof tops. Accordingly, this study was mainly focused on buildings with self-supporting lattice towers in this context.

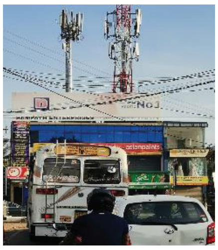
Usually, self-supporting lattice towers are installed on a newly placed beam system on rooftops to ensure transferring additional stresses from rooftop towers directly to columns of host structures. Possibility of occurring drastic increases in stresses and possibility of occurring stress reversal with uplift forces from rooftop towers are the reasons for not installing rooftop towers on existing roof beams and slabs in general. Figure 2 shows a photograph of a typical beam system used on a rooftop in this regard.

Figure 2 – Structural Beam Arrangement of a Rooftop Tower

The general belief is columns have some additional structural capacity over their desirable capacities due to the standard design approaches, hence columns would be able to withstand these additional stresses. That is the reason for allowing the installation of rooftop towers by structural designers after reviewing existing designs as per the present regularity process in most of the cases. However, as the loading patterns that develop due to rooftop tower installation significantly differ from conventional loading patterns that exist in columns of a typical building (development of axial tension in columns is highly possible when a rooftop tower is installed, which very rarely occurs in columns of a typical building).

Hence, columns of such a building may subject to non-conventional primary and secondary stress conditions causing structural concerns. Considering this situation, a detailed field survey was carried out on buildings where rooftop towers exist on rooftops. Table 1 shows a summary of the results of that survey.

Table 1 - Details of Site Survey of Rooftop Towers

Site Location	Building details and defects
Site No 01 	Story – G+2 Tower height – 21 m Problem – Cracks in columns
Site No 02 	Story – G+1 Tower height – 18 m Problem – Cracks in columns and walls
Site No 03 	Story – G+1 Tower height – 18 m Problem – No cracks identified
Site No 04 	Story – G+2 Tower height – 12 m Problem – Cracks in visible in columns and walls

The number of sites that were considered for the survey was minimal due to the reluctance of site owners to allow for such a survey and restrictions imposed by the telecommunication operators, who are the owners of these towers. Any onsite structural testing of members of buildings were not able to be performed due to reluctance of site owners and mobile operators to perform such tests.

As per the results of the survey, visible cracks in columns were identified at 3 sites, and the worst affected site out of these sites was site no. 04. Clear visible cracks had been observed in certain columns between the 1st floor level and the roof slab level in this site. The photographs of observed cracks and a graphical representation of crack locations in 3D structural model of the building are shown in Figure 3 and Figure 4, respectively. A photograph of the tower at site 04 is shown in Figure 5. Any visible cracks were not observed in any other columns at this site.

(a) Cracks in the column at corner C4 column (Site No 04- 1st floor)

(b) Cracks in the column at C3 column (Site No 04- First Floor)

Figure 3 - Locations of Cracks Reported at Site No 4

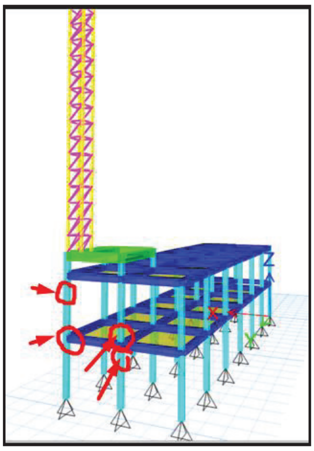


Figure 4 – Graphical Representation of Crack Locations in Columns at Site No 04

Figure 5 – Photograph of the Tower at Site No 04

Based on the results of this site survey, it was rationally suspected that additional loads from rooftop towers would be one main probable reason for these cracks in columns. Therefore, it was decided to perform a detailed analysis to study additional forces and stresses developed in a building due to installation of a rooftop tower by selecting a suitable case study. Due to considerable number of reported defects and the general layout of the building, site no. 04 was selected in this regard. This building is a rectangular building having a size of 19.2m x 6m and column spacing of 3.2m x 3m as shown in Figure 6. Therefore, it would be a good selection to represent this type of common low-rise buildings in the country. However, it was not able to find structural drawings of the building. But, as per information obtained from the owner, the Grade of concrete was considered as Grade 25 and reinforcement of columns were considered as 4T12.

Accordingly, for the analytical study, firstly a 3D analytical model of the building with a 12-meter rooftop tower was developed using ETABS[7] general purpose structural analysis software simulating actual case present at site No 04 (Location 01 of Figure 6). Further, to assess the variation of additional loads transferred to columns based on the location of the tower on the rooftop, another five (05) locations (as marked in Figure 6) were considered for this study. Separate analytical models were prepared for each such case changing the location of the tower on the roof top. Typical models of such cases are shown in Figure 7. Also, a separate model without a rooftop tower was developed to simulate the building prior to installation of the rooftop tower.

Masonry in fill walls available in the site was not considered for this analysis as those were not structural elements of the building.
Geometric details of the building considered for this analysis are as follows;

- Type of building: Residential Building
- Building dimensions: 19.2 m x 6 m
- No of floors: G+1 (with a roof slab)
- Slab thickness: 125 mm
- Column size: 225 mm x225 mm
- Beam size: 225mm x 300 mm
- Floor to floor height: 3 m

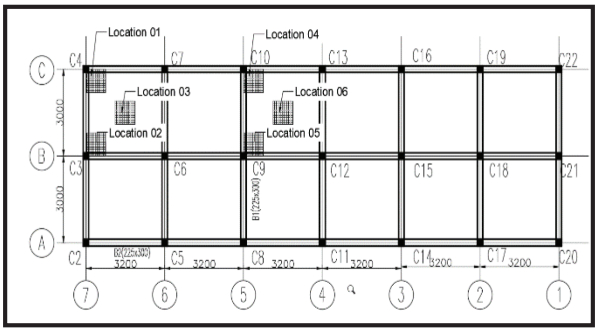


Figure 6 –Locations of the Tower Considered for Analytical Study in Plan View

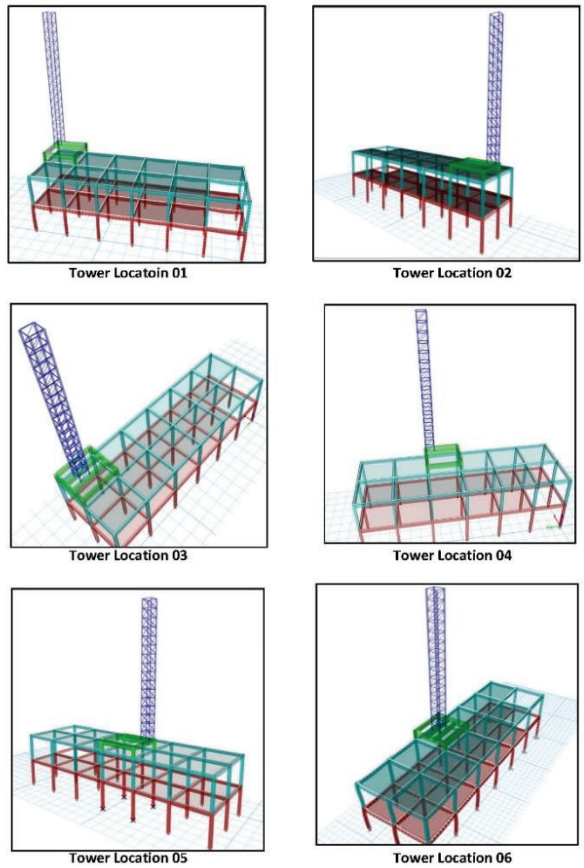


Figure 7 – Analysis Models with the Different Tower Locations

It was decided to perform liner elastic analysis using ETABS software for each of the above cases as the main purpose of this study was to study the additional stress on columns due to the rooftop tower installation under the ultimate design condition. Beams and columns of models were defined as frame elements according to the sizes of those elements in the actual building. Slab panels were introduced as area elements in models. The rooftop tower was also defined at appropriate locations in respective models as per member sizes of the actual tower available at site 04. The material properties of concrete elements were assigned as per properties of Grade 25 concrete. Material properties for tower members were assigned as per properties of Grade S275 steel. The support condition of all columns at the foundation level of the building was considered as a pin support.
For the development of analytical cases and design verification it was decided to use BS8110:1997 [8] and CP3: Chapter V[9] (for wind load calculations) as all of these structures would have been initially designed as per above codes, even though now execution of structural designs are gradually changing to Euro codes .

Loading that were considered for all analytical models are as follows:

Gravity Loads

- Finishes and Partitions – 2.5 kN/m^2 (Considering half walls and other floor finishes)
- Ceiling & Services – 0.5 kN/m^2
- Live Loads – 2.5 kN/m^2

Lateral Loads

- Wind Speed – 33 m/s

As the considered building is in the Colombo district, a wind speed of 33 m/s [10] was considered for the calculation of wind loads on the building and the tower. Wind loads in the rooftop tower and on the building were calculated as per the standard approach CP 3: Chapter V. Wind load on tower was calculated considering an antenna wind shield area of 10m^2 at the top level of the tower. Further, wind load on the body of the tower was calculated by considering solidity ratios of induvial panels as per the standard practice. Calculated loads were applied to respective models as nodal loads.

The following load combinations were considered as per BS8110:1997.

1. 1.4 G_k +1.6 Q_k
2. 1.0 G_k + 1.4 W_k
3. 1.2 (Q_k + G_k + W_k)

(Q_k - Impose loads, G_k - Dead loads, W_k- Wind loads)

However, under wind load combinations, eight (08) wind directional conditions shown in Figure 8 were considered to identify the worst-case scenario.

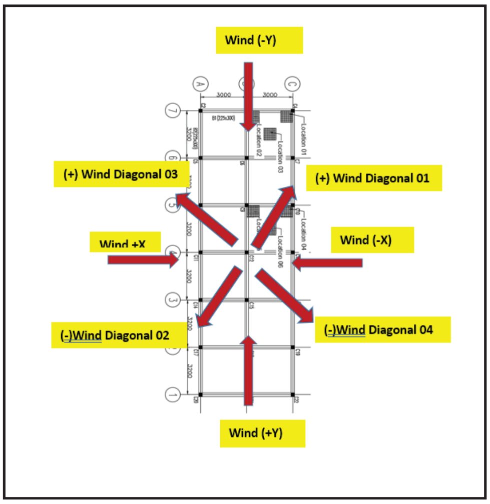


Figure 8 – Locations of the Tower for Analytical Study

Accordingly, 07 load cases and 17 different load combinations were considered with 8 directional combinations as listed below for each analytical model;

01. Load Cases

- Case 01 – Dead
- Case 02 – Live
- Case 03 – Wind X
- Case 04 – Wind Y
- Case 05 – Superimposed Dead
- Case 06 - Wind Diagonal Positive
- Case 07 – Wind Diagonal Negative

02. Load combinations

- Com 01- 1.4G_k+1.6Q_k
- Com 02- 1.2G_k+1.2Q_k+1.2W_x
- Com 03- 1.2G_k+1.2Q_k+1.2W_y
- Com 04- 1.2G_k+1.2Q_k+1.2$W_{(Diag.1)}$
- Com 05- 1.2G_k+1.2Q_k+1.2$W_{(Diag.2)}$
- Com 06- 1.2G_k+1.2Q_k+1.2$W_{(Diag.3)}$
- Com 07- 1.2G_k+1.2Q_k +1.2$W_{(Diag.4)}$
- Com 08- 1.2Gk+1.2Q_k +1.2W_{-x}
- Com 09- 1.2G_k+1.2Q_k+1.2W_{-y}
- Com 10- 1.0G_k + 1.4W_x
- Com 11- 1.0G_k + 1.4W_y
- Com 12- 1.0G_k + 1.4W_{-x}
- Com 13- 1.0G_k + 1.4W_{-y}
- Com 14- 1.0G_k + 1.4$W_{Diag.1}$
- Com 15- 1.0G_k + 1.4$W_{Diag.2}$
- Com 16- 1.0G_k + 1.4$W_{Diag.3}$
- Com 17- 1.0G_k + 1.4$W_{Diag.4}$

3. Results and Discussion

Under additional loads from a rooftop tower, two main structurally critical conditions can arise in columns of a building.

1. Increase of axial compression and bending moment of columns
2. Development of axial tension and bending moments in columns

In case of condition 1, the most critical locations to be assessed would be the foundation level and first floor level of the building. In case condition 2, the critical location would be just below the rooftop level, as any axial tension that would develop would diminish with vertical loads from slabs.

3.1 Assessment of Axial Compression and Bending Moment of Columns with Rooftop Tower Installation

Table 2 shows column forces under initial condition (without rooftop tower) at foundation level. P, M2 and M3 of Table 2 represent Axial force, Maximum bending moment in local direction 2 and Maximum bending moment in local direction 3, respectively.

Table 2 - Column Loads at Foundation Level under Initial Condition (without Rooftop Tower)

Column	Load Comb.	P (kN)	M2 (kNm)	M3 (kNm)
C2	Com 01	-150.10	0	0
C3	Com 01	-247.10	0	0
C4	Com 01	-150.10	0	0
C5	Com 01	-243.00	0	0
C6	Com 01	-311.00	0	0
C7	Com 01	-243.00	0	0
C8	Com 01	-229.4	0	0
C9	Com 01	-298.10	0	0
C10	Com 01	-229.00	0	0

Table 3 shows column forces at foundation level in cases where presence of the rooftop tower at different locations (Location 1 to 6). In both tables, maximum reported compressive forces were tabulated with respect to each column with respective load combinations. The highest compressive load of 389.53kN was reported in column C9. This was reported in the model, where the tower was located at Location 05. The relevant load combination for that case was Combination 06 ($1.2G_k+1.2Q_k+1.2W_{(Diag.3)}$) (refer to Table 3).

Table 3 - Column Loads with Tower Case at Foundation Level

Tower Loc.	Column	Load Comb.	P (kN)	M2 (kNm)	M3 (kNm)
01	**C3**	Com 01	-293.28	**0**	**0**
	C4	Com 01	-278.90	**0**	**0**
	C6	Com 01	-324.81	**0**	**0**
	C7	Com 01	-289.05	**0**	**0**
02	**C3**	Com 06	-353.96	**0**	**0**
	C4	Com 10	-230.32	**0**	**0**
	C6	Com 13	-336.39	**0**	**0**
	C7	Com 01	-258.59	**0**	**0**
03	**C3**	Com 06	-297.29	**0**	**0**
	C4	Com 04	-225.29	**0**	**0**
	C6	Com 05	-341.84	**0**	**0**
	C7	Com 07	-300.01	**0**	**0**
	C9	Com 12	-331.62	**0**	**0**
	C10	Com 04	-338.87	**0**	**0**
	C12	Com 01	-314.51	**0**	**0**
04	C13	Com 13	-275.36	**0**	**0**
05	**C9**	**Com 06**	**-389.53**	**0**	**0**
	C10	Com 10	-291.97	**0**	**0**
	C12	Com 11	-326.58	**0**	**0**
	C13	Com 01	-247.36	**0**	**0**
06	C9	Com 06	-332.64	**0**	**0**
	C10	Com 04	-284.90	**0**	**0**
	C12	Com 05	-331.97	**0**	**0**
	C13	Com 07	-285.17	**0**	**0**

The comparison of maximum column loads of with and without tower cases at foundation level of columns are presented in Table 4.

Table 4 - Comparison of Increment of Compressive Column Loads at Foundation Level under Different Tower Locations

Tower Loc.	Column	Maximum compression with tower (kN)	Maximum compressive force without tower (kN)	% of increment in Compression
01	C3	-293.28	-247.10	18.69%
	C4	-278.90	-150.10	**85.81%**
	C6	-325.10	-311.80	4.27%
	C7	-287.20	-243.00	18.19%
02	C3	-353.96	-247.10	43.25%
	C4	-230.32	-150.10	53.44%
	C6	-336.39	-311.80	7.89%
	C7	-258.59	-243.00	6.41%
03	C3	-297.29	-247.10	20.31%
	C4	-225.29	-150.10	50.09%
	C6	-341.84	-311.80	9.63%
	C7	-300.01	-243.00	23.46%
	C9	-331.62	-298.10	11.24%
	C10	-338.87	-229.00	47.98%
	C12	-314.51	-300.00	4.84%
04	C13	-275.36	-231.60	18.90%
05	C9	-389.53	-298.10	30.67%
	C10	-291.97	-229.00	27.50%
	C12	-326.58	-300.00	8.86%
	C13	-247.36	-231.60	6.80%
06	C9	-332.64	-298.10	11.59%
	C10	-284.90	-229.00	24.41%
	C12	-331.97	-300.00	10.66%
	C13	-285.17	-231.60	23.13%

The main reason for the increase of column loads was the behaviour of a tower as a vertical cantilever against wind loads, causing the increase of compressive loads either in one (under diagonal wind cases) or two columns (under wind in X or Y directions) in leeward side while causing uplift in columns in windward side.

The highest percentage increase of compressive load in a column due to tower installation was reported in column C4 with the tower Location 01 case. Incidentally, the case 01 represents the condition where tower was really available at site 04. The percentage increase of compressive load in column C4 in that case was high as 85.8% from initial condition (without tower case-refer to Table 4). When it considers column loads in general, relatively high percentage increases were reported in cases where the tower was located close to a corner or edge of the building (cases 1,2,3 & 4).

Based on these reported column forces and bending moments, design verifications of columns were performed for all cases separately as per BS8110:1997. Table 5 presents the required reinforcement amounts at foundation level under initial condition, and with the availability of the tower at respective locations. According to the results of design verifications, the reinforcement requirements of columns at foundation level with the presence of rooftop towers could still be satisfied with nominal amount of reinforcements, even though considerably high percentage increases of compressive forces were reported due to installation of rooftop towers as discussed previously.

Table 5 – Comparison of Reinforcement Requirements of Columns at the Foundation Level

Tower Loc.	Column	Critical load combination considered for design verification	$A_{s\,req}$ as per initial condition (without tower case) (mm²)	$A_{s\,req}$ with the presence of tower (mm²)
01	C3	Com 06	202.5	202.5
	C4	Com 04	202.5	202.5
	C6	Com 01	202.5	202.5
	C7	Com 07	202.5	202.5
02	C3	Com 06	202.5	202.5
	C4	Com 10	202.5	202.5
	C6	Com 13	202.5	202.5
	C7	Com 01	202.5	202.5
03	C3	Com 06	202.5	202.5
	C4	Com 04	202.5	202.5
	C6	Com 05	202.5	202.5
	C7	Com 07	202.5	202.5
04	C9	Com 12	202.5	202.5
	C10	Com 04	202.5	202.5
	C12	Com 01	202.5	202.5
	C13	Com 13	202.5	202.5
05	C9	Com 06	202.5	202.5
	C10	Com 10	202.5	202.5
	C12	Com 11	202.5	202.5
	C13	Com 01	202.5	202.5
06	C9	Com 06	202.5	202.5
	C10	Com 04	202.5	202.5
	C12	Com 05	202.5	202.5
	C13	Com 07	202.5	202.5

As already highlighted, assessment of increment of the axial compression and bending moment of columns at 1st floor level was also carried out. Accordingly, Table 6 shows reported design axial forces, bending moments and required amount of reinforcements at first floor level under the initial condition (without tower case). P, M2 , M3 and A_{sreq} of Table 6 represent Axial force, Maximum Bending moment in local direction 2, Maximum Bending moment in local direction 3 and the amount of reinforcements required as per design verifications, respectively. Table 7 shows the same data of columns at 1st floor level with the availability of the rooftop tower at respective locations on the rooftop.

Table 6- Design Inputs and Reinforcement Requirements of Columns at 1st Level Under the Initial Condition

Column	Load Comb.	P (kN)	M2 (kNm)	M3 (kNm)	$A_{s\,req}$ (mm²)
C2	Com 03	-123.70	-5.50	3.50	202.5
C2	Com 12	-97.30	-3.30	9.40	202.5
C3	Com 03	-204.40	-4.10	-0.02	202.5
C3	Com 12	-145.60	-1.80	8.00	202.5
C4	Com 03	-123.60	-5.50	-3.50	202.5
C4	Com 12	-87.00	-3.20	4.10	202.5
C5	Com 11	-143.30	-1.00	1.40	202.5
C5	Com 12	-118.70	0.50	8.20	202.5
C6	Com 11	-158.30	1.90	0.01	202.5
C6	Com 13	-158.00	0.30	8.00	202.5
C7	Com 13	-143.90	2.20	-1.40	202.5
C7	Com 10	-148.70	0.60	-8.20	202.5
C8	Com 11	-135.20	-1.60	1.40	202.5
C8	Com 8	-193.10	-0.10	8.20	202.5
C9	Com 13	-150.90	1.60	0.02	202.5
C9	Com 10	-150.90	-0.01	-8.00	202.5
C10	Com 11	-135.00	-1.60	-4.00	202.5
C10	Com 10	-140.20	-0.10	-8.2	202.5

Table 7 - Design Inputs and Reinforcement Requirements of Columns at 1st Level with the Availability of Rooftop Tower at Respective Locations

Tower Loc.	Col.	Load Com.	P (kN)	M2 (kNm)	M3 (kNm)	A_{sreq} (mm²)
01	C3	Com 12	-240.00	-1.87	20.0	556.88
	C4	Com 12	-214.40	-1.48	-18.47	556.88
	C6	Com 12	-175.95	0.45	17.29	556.88
	C7	Com 10	-179.90	3.06	-15.64	506.25
02	C3	Com 12	-255.07	-1.93	19.47	556.88
	C4	Com 10	-207.75	-1.43	-19.14	556.88
	C6	Com 12	-179.78	0.54	17.20	506.25
	C7	Com 12	-178.58	2.95	-15.61	506.25
03	C3	Com 12	-211.14	-1.95	18.75	506.25
	C4	Com 10	-172.90	-1.73	-18.00	506.25
	C6	Com 12	-220.72	0.44	16.58	455.63
	C7	Com 10	-222.15	2.67	-15.05	506.25
04	C9	Com 12	-244.88	-0.03	13.29	253.13
	C10	Com 10	-251.60	0.56	-11.80	253.13
	C12	Com 12	-170.66	0.13	12.30	253.13
	C13	Com 10	-168.16	0.86	-11.58	253.13
05	C9	Com 12	-244.88	-0.08	12.72	227.81
	C10	Com 10	-251.60	0.59	-12.52	227.81
	C12	Com 12	-173.95	0.19	2.22	227.81
	C13	Com 10	-166.26	0.78	-11.59	227.81
06	C9	Com 12	-215.98	-0.07	12.53	227.81
	C10	Com 10	-211.05	0.25	-11.78	227.81
	C12	Com 12	-214.96	0.11	12.14	227.81
	C13	Com 10	-210.27	0.46	-11.45	227.81

As per data in Table 7 and Table 8, as expected, increase of design axial forces and bending moments in columns with the presence of the rooftop tower from the initial condition (without rooftop tower case) can be observed. Accordingly, amount of reinforcement requirements too has increased in adjacent columns under all six tower locations from the initial condition. For the comparisons, reinforcement requirements of columns at 1st floor level in the initial condition (without tower case) and with the presence of rooftop tower at different locations on the rooftop are presented in a common table as Table 8.

Table 8- Comparison of Reinforcement Requirements of Columns at 1st Floor Level

Loc.	Col	With Tower		Without Tower		Increm. of A_s (mm^2)
		Load Com.	A_{sreq} (mm^2)	Load Com.	A_{sreq} (mm^2)	
01	C3	Com 12	556.88	Com 12	202.50	354.38
	C4	Com 12	556.88	Com 12	202.50	354.38
	C6	Com 12	556.88	Com 12	202.50	354.38
	C7	Com 10	506.25	Com 10	202.50	303.75
02	C3	Com 12	556.88	Com 10	202.50	354.38
	C4	Com 10	556.88	Com 12	202.50	354.38
	C6	Com 12	506.25	Com 12	202.50	303.75
	C7	Com 12	506.25	Com 10	202.50	303.75
03	C3	Com 12	506.25	Com 12	202.50	303.75
	C4	Com 10	506.25	Com 12	202.50	303.75
	C6	Com 12	455.63	Com 12	202.50	253.13
	C7	Com 10	506.25	Com 10	202.50	303.75
04	C9	Com 12	253.13	Com 10	202.50	50.63
	C10	Com 10	253.13	Com 10	202.50	50.63
	C12	Com 12	253.13	Com 10	202.50	50.63
	C13	Com 10	253.13	Com 10	202.50	50.63
05	C9	Com 12	227.81	Com 10	202.50	25.31
	C10	Com 10	227.81	Com 10	202.50	25.31
	C12	Com 12	227.81	Com 10	202.50	25.31
	C13	Com 10	227.81	Com 10	202.50	25.31
06	C9	Com 12	227.81	Com 10	202.50	25.31
	C10	Com 10	227.81	Com 10	202.50	25.31
	C12	Com 12	227.81	Com 10	202.50	25.31
	C13	Com 10	227.81	Com 10	202.50	25.31

Accordingly, significant increase in the amount of reinforcements to cater for additional compressive forces and bending moments in columns at 1st floor level due to rooftop tower installations were reported when the tower was located at Locations 01, 02 or 03. When one compares the required reinforcement amount in relevant columns under these cases (cases where the rooftop tower is located at Locations 01,02 or 03) with available reinforcement amount in such columns (4T12 = 452mm²), it has exceed by 23% in worst cases (refer data relevant to columns of Locations 01 and 02 of Table 8).

In case of locating the tower at Locations 04, 05 or 06, necessary increments of reinforcements are comparatively small. Especially, in case of locating the tower at Location 05 or 06, necessary increments are marginal and it can be satisfied by the available reinforcement amounts reported in columns (4T12 = 452mm²). The considerable increment of axial compressions and bending moments in corner and edge columns, when a rooftop tower is located at a corner or an edge of the building would be the probable reason for this relatively large increment of reinforcement requirements in these cases.

3.2 Assessment of development of axial tension and bending moment of columns with rooftop tower installation

The most likely position of reporting axial tension in columns would be just below the rooftop level as minimum compressive loads due to dead and imposed loads would be at that level. Accordingly, column loads were checked at just below the rooftop level. Table 9 shows reported maximum tensile forces in columns at that level with the availability of rooftop tower at different locations. Relevant maximum bending moments that were considered for design verification are also presented in the same table (in 5th column of Table 9).

Table 9 - Required Tension Lap Length and Required Steel and Second Floor Level

Tower Loc.	Col.	Load Comb.	Tensi. Force (kN)	M (max) kNm	A_{sreq} (mm^2)	Tension Lap (mm)
01	C3	Com 10	16.8	-5.9	38.44	55.7
	C4	Com 15	89.23	-2.54	205.95	597.13
	C7	Com 10	8.30	-1.90	18.80	28.30
02	C3	Com 17	66.51	-7.31	152.21	441.31
	C4	Com 12	49.04	-4.21	112.51	325.35
03	C3	Com 17	8.35	-5.43	19.10	55.38
	C4	Com 15	35.35	-3.20	80.89	234.54
04	C10	Com 15	67.34	1.86	154.10	446.76
	C13	Com 11	9.78	-2.53	22.40	45.41
05	C9	Com 17	44.29	1.79	101.40	205.68
	C10	Com 12	27.87	-3.84	63.71	184.90
06	C10	Com 15	12.72	-1.99	29.10	84.40
	C13	Com 16	12.10	-1.93	30.00	84.39

Accordingly, amounts of reinforcements required to withstand under these direct tensions and bending moments were calculated as per BS8110:1997 and presented in 6th column of Table 9. The highest tension was reported in column C4 (under tower Location 01 case) and that value was 89.23kN. As per Table 9, tensile forces were reported in three columns of the relevant slab panel with the positioning of the tower at Location 1. In the case of placement of the tower at Locations 02 and 04, considerably larger tensile forces of 66.51kN (in column C3) and 67.34kN (in column C10) were also reported in certain columns.
All these cases are relevant to cases where the tower has been located at a corner or at an edge of the building. However, when the reinforcement requirements were checked to resist these tensile forces as per BS8110:1997, nominal amount

reinforcement (4T12) would still be sufficient under the ultimate limit state in all cases.

Lap lengths that would be provided for column reinforcements may become another concern under the development of tension in columns because, usually, compression lap lengths are provided for reinforcement of columns. Accordingly, the minimum theoretical lap length requirements for each column with reported tensile forces and bending moments were also calculated and tabulated in Table 9 (calculations were performed as per Chapter 3.12 of BS8110:1997).

As per Table 9, it is clear that certain columns would require longer lap lengths than the nominal compression lap length of 300 mm as specified in BS8110:1997 with development of uplift when the rooftop tower is located at Locations 01, 02 and 04. For example, the lap length requirement of column C3 under location 01 case is as high as 597 mm, which is almost two times higher than the compression lap length requirement of 300 mm. In studies done by Fegal et al. (2023) and Melek and Wallace (2004) highlighted that insufficient lap length is one major cause of deboning in concrete elements. Therefore, insufficient lap lengths may lead to cracks in columns under uplifting loads with a rooftop tower at Locations 01, 02 or 04 in an actual building.

3.3 Interpretation of results relevant to field observations

If the actual situation of site no. 04 is considered, which leads to this study, C3 and C4 are the columns where a considerable amount of cracks has been reported. Also, these cracks in columns were reported between first floor and roof slab. Incidentally, insufficient compression capacities (with relevant bending moments) with provided amount of reinforcements (4T12) were reported in same columns as per this analysis (refer Table 7).

Further, longer lap length requirement was also reported in column C4 with respect to this tower location (Location 1) in the same zones of the respective columns (refer Table 9). Hence, as per analytical results, these columns would have been subjected to overstressing under compression, and excessive bond stresses at lap joints with uplift forces cyclically with directional changes of wind from the time where rooftop tower had been installed.

Therefore, combining the effects of cyclic overstressing in respective columns, and at lap joints of respective columns would lead to significant cracks observed above 1st floor in columns C3 and C4 at Site No. 4. Unfortunately, it was not able to perform any onsite testing of columns to further verify this, due to reluctance of site owners. Hence, urgent attention would be required to repair these cracks in columns and strengthening of columns at that site (site no 04).

As per results discussed in sections 3.1 and 3.2, installation of the rooftop tower at Location 05 or 06 would only create a marginal increase of compressive or tensile forces of relevant columns. According to design verifications, increase of these compressive and tensile forces would not exceed both compressive capacity with provided nominal reinforcements of columns (4T12) or bond stress under compressive lap length of 300mm. Therefore, most appropriate locations to recommend the installation of a rooftop tower, considering the structural performance of the columns of this type of building, would be locations 05 or 06 or any other similar location on the rooftop.

4. Conclusion

This study focused on the investigation of structural effects on columns of an existing building due to the installation of rooftop towers through a selected case study based on a field survey.

As per the analytical study performed by selecting a case where cracking of columns was reported after installation of a 12m rooftop tower, it was noted that location of rooftop tower greatly affecting on amount of additional forces (both uplift and compressive forces) that would be transferred to relevant columns of the host structure. In certain cases, such forces exceeded the design capacities of columns, and bond stresses of lap joints of columns especially with uplifting stresses causing axial tension in columns. This situation was especially reported when the tower was located closer to a corner or edge of the building. The exceedance of design capacity of certain columns in the considered building was found to be high as 23% in the worst case (relevant to Location 01, which simulates the actual condition of site 04). Further, required lap length was reported as high as twice the compressive lap length in the same case due to uplift forces generated by roof top tower installation.

Relatively, small vertical loads (which would not be sufficient to diminish axial tension due to behaviour of rooftop tower) and relatively larger bending moments of perimeter columns would be the probable reason for reported overstressing of such columns, when a rooftop tower is located close to the perimeter of the building. However, inadequacy of reinforcements or insufficient lap lengths in columns were not reported when tower is located at an interior location of the building represented by Locations 05 and 06.

As the selected building for this study is a typical rectangular building, in general, it may be possible to generalize findings of this study for this type of buildings. Hence, when installing a rooftop tower on this type of building, it is advisable to locate it close to the middle of the building to minimise excessive stresses in nearby columns. However, usually site owners try to locate rooftop towers at a corner of a building, considering the effective utilization of space for rooftops for other operational purposes. But, this cannot be recommended as per findings of this study.

Nevertheless, it is always advisable to go for a case specific analysis prior to installation of a tower through a qualified structural engineer by adhering to present TRCSL guidelines to ensure avoidance of excessive stresses in columns with the installation of a rooftop tower as per results of this study.

References

1. https://www.trc.gov.lk.

2. https://economictimes.indiatimes.com /nation-world/cyclone-phailin-leaves-a-trail-ofdestruction/cyclone-phailin-aftermath.

3. Gunathilaka, A. M. L. N., “Structural Effects on Existing Buildings due to installation of Rooftop Towers”, Annual Transactions of IESL (2012 March) pp. (112-119), The Institution of Sri Lanka.

4. Aseem, F., & Quadir, A., “Effect of Rooftop Mounted Telecommunication Tower on Design of the Building Structure”, International Journal of Engineering Technology, Volume No. 04 (2017), Issue No 11.

5. Malviya, S., & Jamle, S., ”Determination of Optimum Location of Rooftop Telecommunication Tower over Multistory Building under Seismic Loading”, International Journal of Advanced Engineering Research and Science, 6(2), pp. 65–73, 2019.

6. Vikaskumar, P., Gore, N. G., Sayagavi, V. G., “Influence of Telecommunication Tower on Response of Host Structure”, International Journal of Student Research in Technology & Management, Volume No. 05 (2013), pp. 489-498.

7. User's Guide ETABS® 2016 Integrated Building Design Software, Computers & Structures, Inc., 2016.

8. BS 8110: Part 1:1997: The Structural use of concrete-code of practice for design and construction. British Standard Institution, London, England.

9. CP3: Chapter V: Part 2 (1972): Code of Basic Data for Design of Buildings: Wind loads.

10. Design of Buildings for High Winds-Sri Lanka 1980, Ministry of Local Government, Housing and Construction.

11. Fayed, A., Mansour, W., Tawfik, T.A., Sabol, P., Katunský, D.,(2023) “Techniques Used for Bond Strengthening of Sub-Standard Splices in Concrete: A Review Study”, Mechanical Behavior and Degradation Processes of Advanced Materials (MDPI), 11(4), 1119 (2023); https://doi.org/10.3390/pr11041119.

12. Melek, M., Wallace, J.W., “Performance of Columns with Short Lap Splices”, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 1034.