CHAPTER 4

DEVELOPMENT OF GENERAL ALGORITHM
TO QUANTIFY ACTUATING STRAIN AND VALIDATION
WITH EXPERIMENTAL VALUES

4.1 METHODOLOGY

Normally structures are designed and constructed based on criteria stipulated in design codes and left unattended till their design life (Senoul Utku, 1998). Nevertheless, a different but prominent scenario is emerging and this relates to short term and high amplitude dynamic loads causing responses much higher than the designed load and occurring for a very short duration. This may be because of cyclones, earthquake, man-made disasters etc. It is not worth designing for this situation as it is for a short period and may not occur frequently. Instead if the system is allowed to have a mechanism by which it can sense, be aware and take action to counter this extra load, the system is saved with marginal extra cost. In this chapter, an attempt is made to counter the unanticipated load occurring for a short period, by incorporating smartness.

The actuators are used to insert deformation into tower elements. Figure 4.1 and 4.2 shows the block diagram of the actuator, sensors and control system provided in one face of the tower chosen. The sensors monitor the response of the towers continuously and feed the information to the PC with control board. Based on the predetermined control algorithms, the controller has been instructed to actuate the actuators, fixed in the leg
members of the towers. The actuators develop the required strains to counteract the excess response developed due to the unexpected excitations. This process will continue, until the unexpected load keeps exceeding the design load.

Figure 4.1  Actuators inserting deformation into tower elements

Figure 4.2  Block diagram of control system
4.2 NEED FOR GENERAL ALGORITHM

The previous chapter presented a certain smartness or adaptive character that can be built into a structure through introduction of integrated sensors, actuators and control algorithms into the structural system as such a structure can resist unanticipated overload to a certain extent. It is also seen that strain introduced in certain members produces actuation force to balance the excess loads. This necessitates determining the number of members to be strained and also the magnitude of strain required on each of the members identified. The effective members in the towers vary at every stage of loading also due to the change in the direction of loading. This process poses a problem in fixing the number of actuators which generate balance strains. To counter this problem, a further analysis is made and resultantly, it is found out that fixing actuators only on all the leg members is effective in balancing the excess loads. To determine the magnitude of balancing force in each leg member an algorithm has been developed. The algorithm thus developed is useful in determining the balancing strain in the leg members of the towers.

4.3 GENERAL ALGORITHM

In this chapter the effectiveness of introducing the strains in the leg members of the towers through actuators which balance the responses of the high amplitude dynamic loads when they exceed the originally intended response of the designed load that are occurring for a very short duration is discussed. Initially the required strains at various actuators are calculated for the static loading conditions and then their effectiveness for dynamic loading is studied through an example tower.

The need to develop an algorithm is to determine the quantum of the strains required at various actuators which are fixed in the leg members of
the towers. A tower with n panels subjected to lateral loading is taken for study. The strains developed due to the design load in the leg members are denoted as 1e1 and 1e3 are in the first panel tension members and 1e2 and 1e4 are in the first panel compression members. Similarly 2e1 and 2e3 are in the second panel tension members and 2e2 and 2e4 are in the second panel compression members and so on. These values are not exceeded to maintain the structural safety. When the design load is increased, these strain values are increase into 1e11 and 1e13 in the first panel tension members 1e12 and 1e14 in the first panel compression members, 2e11 and 2e13 in the second panel tension members 2e12 and 2e14 in the second panel compression members etc. resulting in excess compressive strain of (1e12 - 1e2), (1e14 - 1e4), (2e12-2e2) and (2e14 - 2e4) etc. These additional strains are developed in the compression leg members. To compensate the excess strain an external force is applied through actuators, which develop counter strains in these members as well as tension in leg members. To estimate the strain required to be developed in the leg members by the actuators, 2n set of equal strains, say e is applied in the 2n pairs of columns. Each pair consists of compressive strain –e in the tension leg member and tensile strain e in the compression leg member. Due to the first pair of applied strains, the strain developed in the compression leg members are 1e21 and 1e41, 2e21 and 2e41 etc.. Similarly the strains developed in the compression leg members due to the other pairs of strain, can be determined. Assuming A, B, C and D etc. be the values to be multiplied with the strain value e to obtain the actual strain to be applied in each of the 2n pairs of leg members, the value of A, B, C and D etc. can be determined from the following equations.
\[
\begin{bmatrix}
1e21 & 1e22 & 1e23 & 1e24 & \ldots \\
1e41 & 1e42 & 1e43 & 1e44 & \ldots \\
2e21 & 2e22 & 2e23 & 2e24 & \ldots \\
2e41 & 2e42 & 2e43 & 2e44 & \ldots \\
\ldots & \ldots & \ldots & \ldots & \ldots \\
\end{bmatrix}
\begin{bmatrix}
A \\
B \\
C \\
D \\
\ldots \\
\end{bmatrix}
= 
\begin{bmatrix}
1e^2 - 1e2 \\
1e^4 - 1e4 \\
2e^2 - 2e2 \\
2e^4 - 2e4 \\
\ldots \\
\end{bmatrix}
\]

A * e, B * e, C * e and D * e etc. are the respective strains required in each of the four pairs to balance the excess load.

### 4.4 EXPERIMENTAL SET-UP FOR VALIDICTION OF THE ALGORITHM

A 2-story steel tower is designed and it is not a scale down model of any tower. The structure is designed in such a way that it should resist the load in the lateral direction. The geometrical details of the testing model are as follows.

1. First Panel height : 700 mm
2. Second Panel height : 700 mm
3. Total height : 1400 mm
4. First Panel Column Section : $40 \times 40 \times 6$ mm
5. Second Panel Column Section : $35 \times 35 \times 5$ mm
6. Beam Section : $35 \times 35 \times 5$ mm
7. Bracing Section : $35 \times 35 \times 5$ mm
8. Gusset Plate Thickness : 6 mm
9. Base Plate : $300 \times 300 \times 10$ mm
10. Type of connection : Bolted connection

(8 mm high strength bolts with washer)
The 2-panel tower is fabricated with a suitable provision in the base plate to connect the model with the testing frame as shown in the Photographic view of Figures 4.3 and 4.4. All the elements of the truss model are connected by 8 mm diameter bolts with washers. The structure is fixed firmly with the testing frame using mild steel bolts of 18 mm diameter.

Figure 4.3 Photographic view of the loading arrangement of fabricated tower

Figure 4.4 Photographic view of the measuring devices of strain and loading
The leg members are fixed with a mechanism, so that deformation can be introduced into them. The leg member is cut in the middle and welded with two 6 mm thick plates as shown in Figure 4.5. A gap of 12 mm is maintained between the plates fixed in the tension leg members and without gap in the compression leg members. The gap is filled with 0.1 and 0.2 mm thick core plates Figure 4.6. The 6 mm thick plates are connected with 10 mm bolts at all the four corners.

![Figure 4.5 Mechanism to insert deformation](image)

![Figure 4.6 Core plate](image)

The required deformation in each leg member to balance the excess load is calculated based on the algorithm discussed earlier in the chapter. Eighteen selected members of the towers namely, all the eight leg members, six diagonal bracings and four horizontal bracings are fixed with strain gauges
to measure the strains at various stages of loading. Lateral loads are applied at the top two nodes of one side at the rate of 20 KN, 25 KN, 30 KN, and 35 KN and the strain gauge readings are recorded for each set of loading. These processes are continued with a set of deformations inserted in the leg members through the mechanism developed into them. The experimental result is then compared with the analytical study.

4.5 COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

4.5.1 Analytical Study

In the analytical study, the tower is applied with 20 KN load along the lateral direction at top two nodes and the analysis is carried out by using ANSYS 8.0. The force developed in all the members of the tower strains in the leg members and the deflection in the top node, are recorded. At every stage of the loading, load is increased by 25% to the maximum of 75% and the readings are observed. The strain differences that result in at every stage in the leg members of the tower are calculated. Four sets of unit strain, of which each set comprising unit compressive strain in the tension leg member and unit tension strain in compression leg member are applied. For each set, the strain developed in the four compressive leg members is noted down. These values are applied in equation 4.1 to determine the strains required in the actuators in order to balance the load in excess. From this strain value the deformation required to develop this strain at the actuators are calculated this value is rounded to the value in terms of 0.1 mm because in the experimental setup the minimum possible deformation is 0.1 mm only. Strain values corresponding to this deformation are calculated. The 25% excess load is again analysed with the actuator strain and the balanced member forces thus arrived at are recorded for experimental validation. This procedure is repeated at every stage of increasing the load respectively 25%, 50% and 75%.
4.5.2 Experimental Study

In the experimental study, the fabricated tower is erected in the testing frame. Load cell of 10 t capacity is connected with the loading jack. Eighteen members of the tower, all eight column members, four horizontal bracing and four diagonal bracing are fixed with “Bonded Metal Foil Strain gauge” of gauge length 5 mm. Initially 20 kN load is applied in the top two nodes and the strain gauge reading and the deflection at the top node are recorded. Load is increased to 25% and the readings are taken. Loads are removed. Deformation is inserted in the leg members through the actuation mechanism. Tensile strain is developed in the compression column members by inserting required number of core plates and compressive strain is developed in the tension column members by removing required number of core plates. Now again 25% excess load is applied and readings are noted. This process is repeated for 25%, 30% and 35% excess loads and the readings are noted. The theoretical and experimental values are compared. Figures 4.7, 4.8 and 4.9 are the comparison plots.

Figure 4.7 Comparison of theoretical and experimental values of member forces in bottom column member 1-2 and 7-8 before and after control
**Figure 4.8** Comparison of theoretical and experimental values of member forces in top column member 5-6 and 11-12 before and after control

**Figure 4.9** Comparison of theoretical and experimental values of maximum deflection
4.6 TOWER TAKEN FOR THE STUDY

To examine the effectiveness of smartness based on the algorithm so developed, a 32-member tower under lateral load is chosen. From the design point of view, the members are grouped into design groups namely leg members, horizontal bracing members and diagonal bracing members. The geometry and load details are given in Tables 4.1, 4.2 and Figure 4.10.

Table 4.1 Structural details of the tower

<table>
<thead>
<tr>
<th>Description of Members</th>
<th>Members Chosen</th>
<th>Sectional Area Sq. mm.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Panel Leg</td>
<td>ISA 65 × 65 × 6</td>
<td>744</td>
</tr>
<tr>
<td>Top panel leg</td>
<td>ISA 40 × 40 × 6</td>
<td>447</td>
</tr>
<tr>
<td>Diagonal Bracings</td>
<td>ISA 45 × 45 × 6</td>
<td>507</td>
</tr>
<tr>
<td>Horizontal Bracings</td>
<td>ISA 40 × 40 × 6</td>
<td>447</td>
</tr>
</tbody>
</table>

Table 4.2 Design load details for tower

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Node</th>
<th>Design load in N</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Fx</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>50000</td>
</tr>
<tr>
<td>2</td>
<td>9</td>
<td>50000</td>
</tr>
</tbody>
</table>
Figure 4.10   Two Panel Tower chosen for study

4.7 ALGORITHM TO DETERMINE ACTUATING STRAIN

The general algorithm developed in this chapter is now modified to simplify the strain calculations for this problem. The strain developed due to the design load in the leg members is assumed as e1, e2, e3, e4, e5, e6, e7 and e8 of which e1, e2, e3 and e4 in the left side leg members are tensile strain and e5, e6, e7 and e8 in the right side leg members are compressive strain. These values are not to be exceeded to maintain the structural safety. When the design load is increased, these strain values increases to e’5, e’6, e’7 and e’8 respectively resulting in excess compressive strain of (e’5-e5), (e’6-e6), (e’7-e7) and (e’8-e8). These additional strains are developed in the
compression leg members. To compensate the excess strain, external force is applied through actuators, which develop counter strain in these members as well as tension leg members. To estimate the strain required to be developed in the leg members by the actuators, four set of equal strains, say $e$ is applied in the four pairs of columns. Each pair consists of compressive strain $-e$ in the tension leg members and tensile strain $e$ in the compression leg members. Due to the first pair of applied strain, the strain developed in the compression leg members are $e_5^1$, $e_6^1$, $e_7^1$ and $e_8^1$. Similarly strain developed in the compression leg members due to the other three pairs of strain, can be determined. Let $A$, $B$, $C$ and $D$ be the values to be multiplied with the strain value $e$ to obtain the actual strain to be applied in each of the four pairs of leg members. The value of $A$, $B$, $C$ and $D$ can be determined from the following equations.

$$
\begin{bmatrix}
e_{51} & e_{52} & e_{53} & e_{54} \\
e_{61} & e_{62} & e_{63} & e_{64} \\
e_{71} & e_{72} & e_{73} & e_{74} \\
e_{81} & e_{82} & e_{83} & e_{84}
\end{bmatrix}

\begin{bmatrix}
A \\
B \\
C \\
D
\end{bmatrix} =
\begin{bmatrix}
e'_{5} - e_{5} \\
e'_{6} - e_{6} \\
e'_{7} - e_{7} \\
e'_{8} - e_{8}
\end{bmatrix}

(4.2)

A * e, B * e, C * e and D * e are the respective strains required in each of the four pairs to balance the excess load.

4.8 ANALYSIS OF THE TOWER WITH ACTUATOR

In this analysis the effectiveness of the actuating strains in the leg members of the Tower is calculated by using the algorithm.
The study is carried out by increasing the design load at four stages.

Stage I : 25% increase in design load
Stage II : 35% increase in design load
Stage III : 45% increase in design load
Stage IV : 55% increase in design load

The following are the steps involved in calculating the actuating strain by using the algorithm. The analysis is made using ANSYS 8.0.

- Design load 50 KN is applied to the top two nodes of the tower, and the forces in all the members and the strain of the tower are recorded.
- The stage-I load 25% more than the design load is applied and the readings are taken down.
- The excess strain developed over the design value is calculated.
- Four set of unit strains, unit compressive strain in the tension leg member and unit tensile strain in the compression leg member are applied and the strain developed in the compressive leg members in each set is calculated by using ANSYS 8.0.
- The readings taken are substituted in the equation 4.2 and by solving the equation the values A, B, C and D are obtained.
- A * e, B *e, C * e and D * e are the respective strain required in each of the four pairs to balance the excess load.
• These four set of strains are applied in the leg members of the tower as actuating strains to balance the 25% excess load.

• Analysis is made by applying these sets of strains and the first stage of loadings and the results are compared with the same results obtained for the design loadings.

• Dynamic analysis also made by taking the load as rectangular pulse load using ANSYS 8.0 (Appendix 1).

• Analyses are repeated for all the four stages of loadings and the results obtained are discussed.

4.9 RESULTS AND DISCUSSION

4.9.1 Stage I: 25% increase in Design Load

An increase in the load by 25% more than the design value produces forces higher than the design values in nine members. Based on the calculations as mentioned in this chapter, four pairs of strain are obtained by using the equation 4.2. Strain values of 0.00019117 and -0.00019117 in the two pairs of bottom panel leg members and 0.00002494 and -0.00002494 in the two pairs of top panel leg members are applied through the actuators. These responses by the actuators balance the excess force developed in the leg members and partly balance the other members also. Figure 4.11 shows the status of the forces before and after balancing the force in various members.

Dynamic analysis is also carried out by taking the 25% more than the design load as rectangular pulse load. The natural frequency of the structure is 3.25 Hz. Natural period is 0.3077 Sec. For this analysis the period taken for each rectangular pulse is 0.1 times of the natural period, for Type-I loading (Figure 4.12), and, close to natural period, (0.3 Second) for Type-II loading (Figure 4.15).
1. bottom compression leg member, 2. top compression leg member, 3. top horizontal bracing member, 4. bottom horizontal bracing member, 5. bottom diagonal bracing member, 6. top diagonal bracing member.

**Figure 4.11** Force in the critical members before and after balance (Stage I)

**Figure 4.12** Rectangular pulse load for Type-I loading
Figure 4.13 shows the deflection at the top node of the tower with and without the application of the actuation strain. This figure illustrates the effectiveness of the actuator in controlling the maximum deflection. The actuator need not be active throughout the period but only during the peak magnitude of deflection. Figure 4.14 shows that the actuator is active only for 0.3 Seconds.

**Figure 4.13** Deflection for 25% excess of Type-I loading with and without actuation

**Figure 4.14** Deflection for 25% excess of Type-I loading with and without actuation during critical period
Figure 4.16 shows the deflection at the top node of the tower with and without the application of the actuation strain, taking the period for each rectangular pulse as 0.3 seconds (close to natural period Type-II loading). Figure 4.17 shows the response when the actuator is active only for 2.4 seconds.

![Figure 4.15 Rectangular pulse load for Type-II Loading](image1)

![Figure 4.16 Deflection for 25% excess of Type-II loading with and without actuation](image2)
4.9.2 Stage II: 35% increase in Design Load

To balance the 35% excess load, 0.00044001 and -0.00044001 strain in the two pairs of bottom panel leg members, 0.00006407 and -0.00006407 strain in the two pairs of top panel leg members are introduced through the actuators. The forces in the critical members before and after the application of the actuation are shown in Figure 4.18.

Dynamic analysis is carried out by applying the load (35% in excess of the allowable load) under Type-I Loading and Type-II Loading. Figure 4.19 shows the effectiveness of the control strategy in controlling the maximum deflection of the tower for Type-I loading. Figure 4.20 shows the response of the top node when the actuator is active only for 0.3 seconds.
1. bottom compression leg member, 2. top compression leg member, 3. top horizontal bracing member, 4. bottom horizontal bracing member, 5. bottom diagonal bracing member, 6. top diagonal bracing member

**Figure 4.18** Force in the critical members before and after balance (Stage II)

**Figure 4.19** Deflection for 35% excess of Type-I loading with and without actuation
Figure 4.20 Deflection for 35% excess of Type-I loading with and without actuation during critical period

Figure 4.21 shows the deflection at the top node of the tower with and without the application of the actuation strain, when the Type-II loading is increased to 35% more. Figure 4.22 shows the response of the top node when the actuator is active only by 2.4 seconds.

Figure 4.21 Deflection for 35% excess of Type-II loading with and without actuation
4.9.3 Stage III: 45% increase in Design Load

Strain 0.00071099 and -0.00071099 are applied in the two pairs of bottom panel leg members and 0.00018843 and -0.00018843 are the strains in the two pairs of top panel leg members to balance the 45% excess load. The forces in the critical members before and after the application of the actuation are shown in Figure 4.23.

Dynamic analysis result for the 35% more than Type-I and Type-II loading is shown in Figures 4.24 and 4.26. Figures 4.25 and 4.27 shows the maximum deflection of the top node with and without actuator when actuator is active only for 0.3 Seconds.
1. bottom compression leg member, 2. top compression leg member, 3. top horizontal bracing member, 4. bottom horizontal bracing member, 5. bottom diagonal bracing member, 6. top diagonal bracing member

**Figure 4.23** Force in the critical members (stage III)

**Figure 4.24** Deflection for 45% excess of Type-I loading with and without actuation
Figure 4.25 Deflection for 45% excess of Type-I loading with and without actuation during critical period

Figure 4.26 Deflection for 45% excess of Type-II loading with and without actuation
Figure 4.27  Deflection for 45% excess of Type-II loading with and without actuation during critical period

4.9.4  Stage IV : 55% increase in Design Load

Strain 0.00097598 and -0.00097598 are required to balance excess load of 55% in the two pairs of bottom panel leg members 0.00031281 and -0.00031281 are the strains in the two pairs of top panel leg members. The forces and deflection in the critical members before and after the application of the actuation are shown in Figures 4.28.

Figures 4.29 and 4.31 show the dynamic analysis results for the 55% more than Type-I and Type-II loading. Figures 4.30 and 4.32 show the maximum deflection at the top node with and without actuator when actuator is active only for 0.3 Seconds.
1. bottom compression leg member, 2. top compression leg member, 3. top horizontal bracing member, 4. bottom horizontal bracing member, 5. bottom diagonal bracing member, 6. top diagonal bracing member.

**Figure 4.28** Force in the critical members (Stage IV)

**Figure 4.29** Deflection for 55% excess of Type-I loading with and without actuation
Figure 4.30 Deflection for 55% excess of Type-I loading with and without actuation during critical period

Figure 4.31 Deflection for 55% excess of Type-II loading with and without actuation
Figure 4.32 Deflection for 55% excess of Type-II loading with and without actuation during critical period

4.10 DISCUSSION

These results prove that the strain produced by the actuators can successfully balance the excess force in the leg members. However, during the process the force in some of the bracings exceeds its design values marginally. These excess forces in the bracings are not so predominant and it is worth making the structure smart by introducing strain inducing actuators. The bracings can be redesigned for the changed force or the bracing pattern itself can be changed to withstand the excess load.

The maximum deflection of the tower at various stages of overloading before and after control is illustrated in Figure 4.33. It clearly indicates the effectiveness of the control through reduction in deflection.
Figure 4.33 Deflection at various stages of loading with and without control

4.11 SUMMARY

- Certain active control character can be built in to a structure through deformation into the leg members of towers, such that the structure can resist to certain extent to the unanticipated overload.

- The example presented partially illustrates the possibility and effectiveness of this concept.

- A general algorithm to determine the actuator strain is developed and validated by comparing the analytical study with the experimental results.

- The time history analysis is carried out for two story tower with two types of rectangular pulse loading and the controller reduces the maximum displacement with respect to tower...
without controller by 28.28%, 59.82%, 61.77% and 33.33% for the increase in loading by 25%, 35%, 45% and 55% respectively in case of Type-I loading. In case of Type-II loading the maximum displacement reduces by 14.67%, 48.35%, 38.93% and 27.93% respectively.

- The smartness that can be in-built into the system has certain limitations and requires alteration in the design of the bracings.
- The same actuators can be effectively utilized to provide control in either of the two directions.
- The results presented above are encouraging and show that this simple control system can be effective for tower structure.