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An Ideal strain gage placement plan for structural health monitoring under seismic loadings

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Abstract. Structural Health Monitoring (SHM) systems can provide valuable information regarding the safety of structures during and after ground motions which can be used by authorities to reduce post-earthquake hazards. Strain gages as a key element play an important role in the success of SHM systems. Reducing the number of required strain gages while keeping the efficiency of SHM system not only can reduce the cost of structural health monitoring but also avoids storage and process of uninformative data. In this study, a method based on performance based seismic design of structures is proposed for ideal placement of stain gages in structures. The robustness and efficiency of the proposed method is demonstrated through installation of strain gages on an Airport Traffic Control (ATC) Tower. The obtained results show that the number of required strain gages decrease significantly.

Keywords: structural health monitoring; strain gage; sensor placement; airport traffic control tower; performance based seismic design

1. Introduction

Seismic damage identification is a challenging issue in the field of Structural Health Monitoring (SHM). Contrary to static and moving loads, seismic loads have a complex and usually unpredictable effects on structures. This characteristic of seismic loads gives a remarkable place to seismic induced damage identification because it can provide valuable information regarding the safety of structures soon after ground motions. Such information can be employed by authorities to reduce post-earthquake hazards. Although SHM systems can significantly elevate safety of structures, the high cost of required equipment has been the main obstacle for wider application of these systems. Sensors are among costly equipment of SHM systems that play significant role in data acquisition. Reduction in the number of employed sensors can directly decrease the total cost of SHM systems. Moreover, by decreasing the number of sensors, the volume of uninformative data can be reduced; consequently, storage of data is decreased and data processing is accelerated. In SHM systems different types of sensors are employed; accelerometers and strain gages are among the most practiced sensors and can be found in all SHM systems. Basically, accelerometers are employed to extract modal properties (i.e., natural frequencies, mode shapes, damping ratios,

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etc.). Modal properties are essential information for vibration-based techniques to localize damage and estimate its severity. Extraction of modal properties from measured response accelerations requires additional data processing efforts which make vibration-based damage identification methods more complicated. On the other hand, strain gages directly display the level of strain in structural components. By assigning appropriate thresholds to the strain levels, SHM systems can assure the safety of structural components. This means that, structural health monitoring through strain gages are more straightforward and fast in comparison to the application of accelerometers. The main drawback of such a SHM system is that special attention must be paid to the optimal placement of strain gages. Otherwise, in addition to the high cost of large number of strain gages, storage and processing of data will be a problematic issue for the SHM system.

Efforts have been made to optimize the number of sensors used in SHM systems. However, as can be seen from the literature, researchers have only studied optimal sensor placement of accelerometers in order to estimate modal properties and less attention has been paid to strain gages. Liu *et al.* (2008) used genetic algorithm for optimal sensor placement. They provided an improved genetic algorithm to maximize obtained data information from spatial lattice structures. The obtained results showed that the proposed method could identify the vibration characteristics of 12-bay plane truss model. Flynn and Todd (2009), by working on Bayes risk theory, provided a global optimality criterion to minimize error types I and II during damage detection process. From the test results, they showed that the placement of sensors depends on performance constraints. Papadimitriou (2004) presented a formula for optimal sensor placement based on the information entropy measure of parameter uncertainty. In the method, two algorithms for prediction of optimal and worst sensors configurations were proposed. Li *et al.* (2004) studied optimal sensor locations for structural vibration measurements based on uniform design method. Skjærbæk *et al.* (1996) used a measured acceleration response time series to localize the damage in seismically excited reinforced concrete structures. They applied their method on a six-story, two-bay frame and tried to find the optimal locations of sensors. They assumed that measurements were performed at top story and ground surface and tried to find the best locations for one or more sensors in between. The obtained results showed placing sensors at lower parts of the structure provided better indication of damage. Xie and Xue (2006) applied hybrid algorithm using improved reduced system and singular value decomposition for obtaining optimal number and locations of sensors for structural health monitoring. For optimal sensor placement, Zhao *et al.* (2013) defined the ratio of contribution of measurement points to a Fisher information matrix, and the damage sensitivity to the measurement noise as sensor placement indices. Through a laboratory test of a steel frame it was shown that measurement points with larger sensor placement indices are more prominent for damage identification and should be selected for sensor installation. Udwadia (1994) used the concept of efficient estimator to decouple optimization problem from identification problem. The proposed algorithm was capable of locating the optimal place for sensors that yielded the best estimate of dynamic parameters (e.g., change in the stiffness). In addition, it provided a solution for optimal installation of new sensors in an equipped system.

As can be seen from the literature, previous studies have tried to address optimal location of sensors (accelerometers) such that the best estimates of dynamic parameters (i.e., modal parameters) are obtained. However, since strain gage placement deals with the determination of most vulnerable structural components, measurement of dynamic parameters alone does not lead to an ideal strain gage placement plan. The difference between optimal strain gage placement and optimal accelerometer placement becomes more evident when the aim of a SHM system is to monitor structural safety against seismic actions. Unlike the other types of damages, the seismic induced damage depends on the seismicity of the area where structures are located. Far-field and near-field earthquake records may impose completely different types of damages to the structural components. This means that, for two completely similar structures but located in two different seismic zones an ideal strain gage placement method should propose two different locations for installation of strain gages and two different numbers of required sensors. However, as the literature show, the current sensor placement methods are incapable of differentiating this important characteristic of seismic actions. Moreover, when it comes to monitoring safety of structures against seismic actions, it is crucial to pay attention to the seismic performance level of structures. For example, a hospital which has a significant role in reduction of post-earthquake problems requires a higher seismic performance level in comparison to a common residential building having completely similar structure to that of hospital and located in the same seismic zone. This means that an ideal strain gage placement plan in terms of number of installed sensors should equip the hospital more densely compared to the residential building so that more structural components can be monitored. This important characteristic of SHM under seismic loadings has not been addressed by existing optimal sensor placement algorithms.

The main aim of this study is to propose a method for ideal strain gage placement in structures when they are subjected to seismic loads. The proposed method does not follow the current trend of optimal sensor placement in which accelerometers are installed in places where the best estimate of modal parameters are obtained. However, the proposed method takes into consideration the seismicity of area where structures are located and includes the influence of their seismic performance level into the ideal strain gage placement plan. The concept of Performance Based Seismic Design (PBSD) is employed for this purpose in which structures can be designed simultaneously for pairs of performance levels and seismic hazards. In the next sections at first the concept of proposed method is presented and then in order to show the robustness and feasibility of the method it is applied to an ATC tower.

2. The proposed method for ideal strain gage placement

In PBSD a structure can be designed for a range of performance levels when subjected to different levels of seismic hazards. The main structural performance levels are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) levels. According to FEMA 356 (2000), the IO performance level is defined as the post-earthquake damage state that remains safe to occupy and essentially retains the pre-earthquake design strength and stiffness of the structure. Moreover, the LS performance level is defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse. The CP is defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse. Seismic activity of a construction site can be defined by different levels of seismic hazards. Design Based Earthquake (DBE) and Maximum Probable Earthquake (MPE) are two levels of seismic hazard that have been proposed by building codes to design structures and control their safety against seismic actions. The DBE is an earthquake defined to have a 90% probability of non-occurrence in a 50-year-exposure period, which is equivalent to a recurrence interval of 474 years. The MPE is an earthquake defined to have a 2-percent probability of being exceeded in 50 years that is expected to occur once in approximately 2500 years. Structural performance levels and seismic hazard levels can be combined appropriately in order to define performance objectives for structures. It is obvious that unlike the conventional design method, PBSD provides this opportunity for practice engineers to design structures considering different performance objectives.

The proposed method for strain gage placement follows a similar concept to that of PBSD. The main reason is that the two important characteristics that were required for an ideal strain gage placement can be supplied by the PBSD. By following the concept of PBSD, strain gages can be placed at locations where structural components do not satisfy the performance objectives. By employing this concept, strain gage placement will depend on the seismic hazard scenarios as well as considered structural performance levels. A direct benefit of such sensor installation concept is that, the number of required strain gages increases when we elevate the target performance levels or raise seismic hazard levels and decreases when we lower them. Therefore, for placement of strain gages, not only the seismic performance level of structures is considered but the seismicity of the area where structures are located is also taken into account.

Fig. 1 depicts the step by step procedure of the proposed ideal strain gage placement method. In the first step, performance objectives that include structural performance levels and seismic hazard scenarios should be determined. This step directly influences the numbers of required strain gages and their locations. Therefore, special attention should be paid for appropriate determination of seismic hazard levels and structural performance levels. For a given structure, seismic hazard scenarios depend on the regional geology and seismology setting of the construction sites. Normally, seismic hazard maps published in building codes are employed to determine seismic hazard levels. However, for special structures like dams and nuclear power plants, in-site investigations must be performed in order to address comprehensively all probable seismic hazard scenarios through deterministic or probabilistic approaches. On the other hand, determination of seismic performance levels for a structure is a subjective matter and often engineering judgment is used in order to define appropriately which structural performance levels would match well with the condition of structures.

The second step for the ideal strain gage placement deals with Finite Element (FE) modelling. At this step, a nonlinear FE model should be established for structures considering all vulnerable structural components. Structural components can be classified into primary and secondary elements. Elements that contribute to the capacity of the structure to resists collapse due to seismic induced forces shall be classified as primary elements and must be included in the FE model. Secondary elements are structural components other than the primary elements and are expected to be able to support gravity loads under maximum structural deformation. There is no obligation to consider secondary elements into the FE model; however, if their seismic behaviour is significant for sensor installation, the established FE model should also include secondary elements. When the FE model of the structure has been established, it should be subjected to a set of ground motions that represent all probable seismic hazard scenarios. Soil type, source-to-site distance, directivity effect and the ratio of Peak Ground Acceleration (PGA) to Peak Ground Velocity (PGV) are important parameters that should be taken into account when earthquake records are selected. Nonlinear time history analysis is known to be the most accurate and in hand technique that can simulate the behaviour of structures when they are under seismic actions. Therefore, this technique is used in the third step for analysing structures. Plastic hinge formations, damage mechanisms and internal forces should be monitored after performing nonlinear analyses. In the fourth step, by employing results of nonlinear time history analyses, usage ratios that are defined as the ratio of demand to capacity of structural components can be calculated for all considered structural performance levels. Finally, structural components that their usage ratios do not satisfy

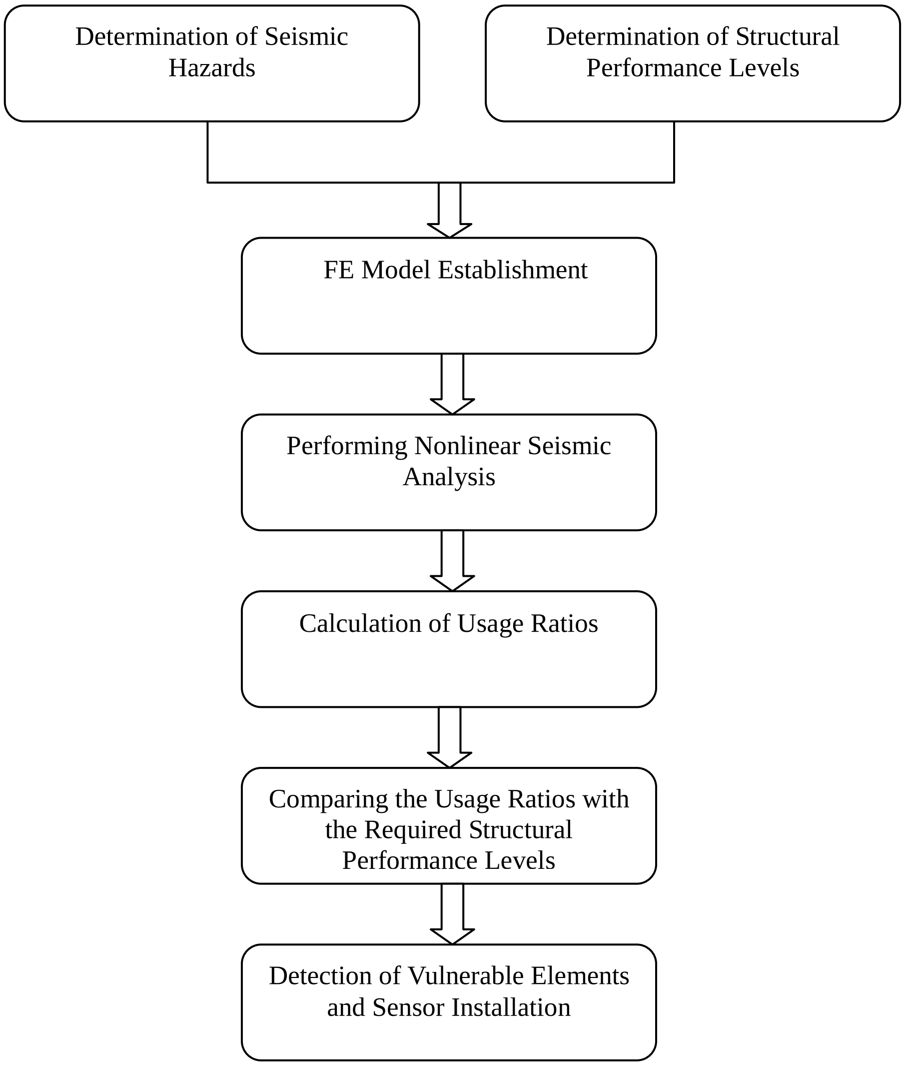

Fig. 1 Step by step procedure for ideal strain gage placement

the requirements of considered structural performance levels are identified as vulnerable elements for strain gage placement.

3. Selected structure

In order to demonstrate the feasibility and robustness of the proposed method, it was applied to the ATC tower of Kerman International Airport, Iran. Fig. 2 displays the geometry of the tower. As can be seen from this figure, the structural system of the tower is divided into two parts. The first part comprises of two concrete shafts, which serve as the lateral load resistance system against wind and seismic loads. The total height of this part is 26.4 m. The thicknesses of concrete walls for exterior and interior shafts are 0.4 m and 0.25 m respectively. Stairs are located between the concrete shafts and a lift is placed inside the interior concrete shaft. The tower has a mat foundation with the thickness of 1.5 m and dimensions of 13 m by 13 m.

The second part of the tower consists of a steel moment resistance frame and is the place where the observation and equipment rooms are located. Steel columns that are connected to each other through steel beams at two different levels transfer gravity and lateral loads of this part to the concrete shafts. The columns have a box girder cross section with constant thickness of 2.5 cm and dimensions of 30×30 cm. The columns settled on 12 radial concrete beams located at level 26.4m. The dimensions of radial concrete beams are 60×30 cm. A 30 cm thick concrete slab connects these concrete beams to each other and creates a rigid diaphragm at level 26.4 m.

The roof of the tower consists of six radial inclined beams with the cross section of IPE27. Twelve I shape girders with the height of 35cm connect the columns at the roof level. The flange width and thickness of these girders are 20 cm and 2 cm, respectively. In addition to the roof level, columns are connected to each other by six I shape curved girders with the height of 30 cm at level 30.1 m. Table 1 shows the reinforcement ratios of concrete walls along the height of the tower. Due to the openings along the height of tower, the concrete walls of exterior shaft are connected to each other via spandrel beams at different levels. All spandrel beams have a rectangular cross section with the dimensions of 50 cm by 40 cm.

Considering soil type and source-to-site distance of the construction site, six natural earthquake records were selected for nonlinear time history analysis. In order to satisfy the PGAs of the construction site for DBE and MPE, earthquake records were scaled to 0.3 g and 0.45 g,

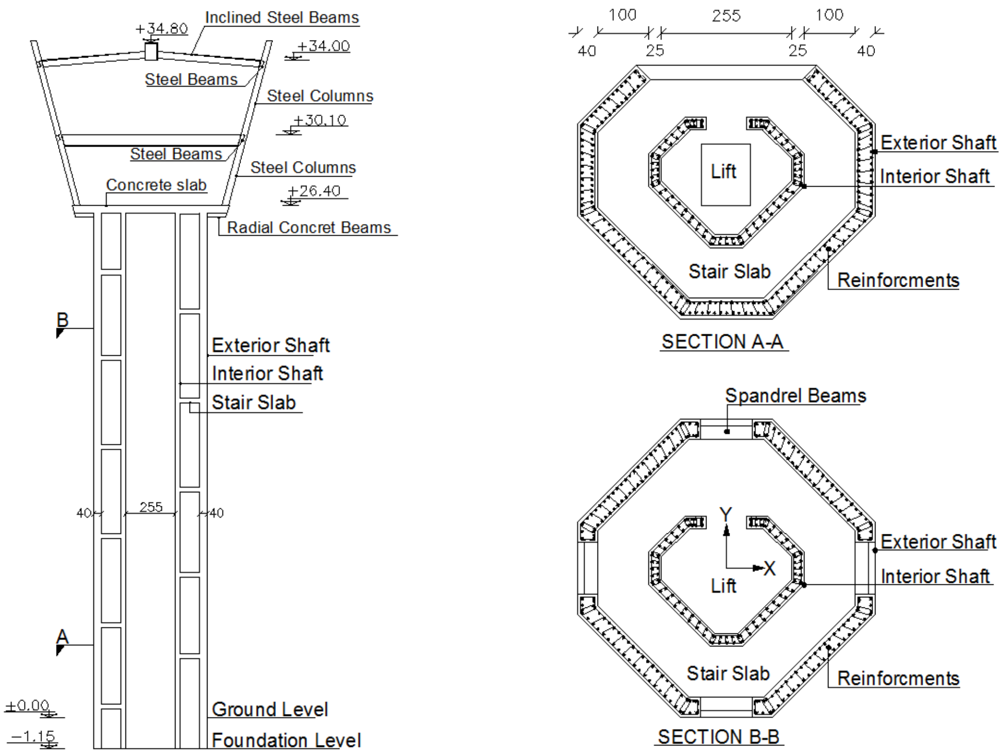

Fig. 2 Longitudinal and cross sectional views of Kerman ATC tower

Table1 Reinforcement ratios along the height of concrete shafts

Height (m)	Exterior Shaft (%)	Interior Shaft (%)
0.0-11.6	1.2	0.84
11.6-21.6	0.79	0.68
21.6-26.4	0.64	0.68

Table 2 Selected earthquake records

No.	Earthquake	Year	Duration(sec.)	PGA(cm/s^2)	PGV(cm/s)	PGA/PGV
1	Park field	1966	10	264.4	14.51	18.2
2	Park field	1966	15	425.7	25.45	16.7
3	San Francisco	1957	10	102.8	4.6	22.3
4	San Francisco	1957	35	83.8	5.05	16.6
5	Helena	1935	15	143.7	7.2	19.9
6	Long Beach	1933	30	95.6	23.7	4.0

Table 3 Categorization of elements into different groups

Group 1:	Circular steel beams at level 34.0 m
Group 2:	Circular steel beams at level 30.1 m
Group 3:	Inclined steel beams at roof level
Group 4:	Radial concrete beams at level 26.4 m
Group 5:	Steel columns between level 26.4 m and 30.1m
Group 6:	Steel columns between level 30.1 m and 34.0 m
Group 7:	Exterior concrete shaft
Group 8:	Interior concrete shaft
Group 9:	Concrete slabs
Group 10:	Spandrel concrete beams between level -1.5 m and 26.4 m

respectively. Table 2 presents the selected earthquake records. Moreover, in order to facilitate localization of strain gages, structural components were categorised into specific groups based on their type and location. As Table 3 shows, totally ten structural groups were created. Each of these groups includes several similar structural components.

4. Determination of performance objectives

Airports have an essential role in the reduction of post-earthquake hazards. Consequently, ATC towers, as inseparable part of airports, must maintain their serviceability during and after seismic events. This implies that they should have a higher seismic performance level in comparison to common buildings (Vafaei *et al.* 2014). Seismic design codes often recommend life safety performance level for common buildings when the seismic hazard is in DBE level. In this study, for strain gage placement in the tower under consideration, two performance objectives are considered as follows:

1) Capability of maintaining Immediate Occupancy (IO) level for the Design Basis Earthquake (DBE).

2) Capability of maintaining Collapse Prevention (CP) level for Maximum Probable Earthquake (MPE).

The first performance objective assures that for moderate earthquakes structural components remain in elastic range so that tower can maintain its functionality. On the other hand, the second performance objective inhibits the tower from collapse when rare earthquakes excite it. These performance objectives aim to identify vulnerable structural components for low and high level of seismic induced damage.

5. Finite element model of the tower

In this study, Perform 3D (Computers and Structures Inc. 2006) software was employed to establish the FE model of the tower. This software programme has been successfully used by researchers to model inelastic behaviour of structures (Zekioglu *et al*. 2007, Chen *et al*. 2010). Fig. 3 shows the created FE model. All openings in the concrete core and slabs were considered in the FE model. However, due to rigidity of foundation, it was assumed that the tower has fixed supports at the foundation level. It was also assumed that slabs remain elastic when the tower is under seismic actions.

Fibre element method was employed to model the inelastic behaviour of concrete shear walls (Zekioglu *et al*. 2007, Chen *et al*. 2010). In this method, nonlinear material properties are assigned to fibre elements and their behaviour are monitored during seismic events. Nonlinear moment-curvature and nonlinear axial force-axial deformation behaviour of shell elements were accounted for in the FE models. Since shear demand was lower than the capacity of the concrete walls elastic behaviour represented the shear deformation of the concrete walls. Table 4 shows nonlinear material properties that were considered in the FE model for concrete and reinforcements. The selected values are in accordance with Federal Emergency Management Agency (FEMA 2000) provisions. Nonlinear behaviour of beams and columns were modelled using plastic hinge technique in accordance with FEMA 356 (2000). The P-Δ effect was included in the nonlinear time history analysis. The first four natural frequencies of the tower obtained from the FE model are presented in Table 5.

Table 4 Modeling parameters for concrete and reinforcements

	Ultimate Compressive Strain	Ultimate Tensile Strain
Concrete	0.005	-
Reinforcements	0.02	0.05

Table 5 Natural frequencies of the tower

	Mode 1(Hz)	Mode 2(Hz)	Mode 3(Hz)	Mode 4(Hz)	Mode 5(Hz)
Type-Direction	Flexural-Y	Flexural-X	Torsional	Flexural-Y	Flexural-X
Frequency	0.96	1.28	2.35	2.51	2.66

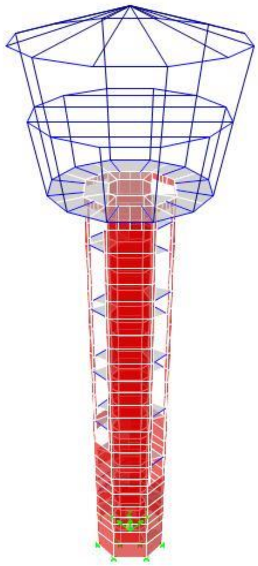

Fig. 3 Finite element model of the tower

6. Results and discussion

Figs. 4 and 5 show shear force and drift demands along the height of the tower when it is subjected to the ground motions scaled to 0.3 g. From Fig. 4 it can be seen that, high values of inter-storey drift demands are concentrated at the levels of observation and equipment rooms. This issue can be related to the lower lateral stiffness of steel moment resistance frames that support the upper part of the tower. High drift demands can be also observed at the lower part of the tower. At these levels concrete walls have large openings in both internal and external shafts which reduce significantly the lateral stiffness of the tower. Shear force distribution along the height of the tower shows that for the lower levels of the tower shear force demands decreases until almost the mid height of the tower. However, after this level, the shear force demand increases along the height of the tower until 30 m height where a sudden decrease in value of shear force demand occurs. Figs. 4 and 5 demonstrates that unlike common buildings ATC towers have a complicated dynamic behaviour that need to be studied more by researchers (Vafaei *et al.* 2014).

Fig. 6 displays modelling parameters and acceptance criteria for the most critical beam of group 2 along with envelops of plastic hinge rotation demands obtained from Nonlinear Time History Analysis (NTHA) when earthquake records were scaled to 0.3 g. The modelling parameters and acceptance criteria are selected in accordance to recommendations of FEMA 356 (FEMA 2000). It should be mentioned that for NTHA in addition to both principal directions, the tower was also excited by angles of 45 degree. Fig. 6 shows that the minimum and maximum plastic hinge rotation demands for the critical beam in group 2 are slightly lower (0.77 rad.) and significantly more (9.9 rad.) than selected value for acceptance criteria of IO level (1 rad.). Therefore, as Fig. 7 also shows, the minimum and maximum usage ratios of group 2 are obtained as equal to 0.77 and 9.9, respectively. A similar procedure was used for other structural groups in order to obtain the maximum and minimum usage ratios for IO and CP levels. Figs. 7 and 8 summaries the obtained usage ratios for IO and CP performance levels. As can be seen from these figures, in comparison to the other groups, structural components in groups 5 and 9 are the most critical elements for strain gage installation. Because even the minimum usage ratios obtained for IO and CP performance levels implies that the least vulnerable elements in these groups will experience severe damage. Therefore, for sensor installation the first priority should be given to these two groups. These figures also display that, group 3 satisfies the requirements of both performance objectives therefore there is no need to install any stain gage on the elements of this group. Although group 4 does not satisfy the requirements of IO performance level, the usage ratios obtained from CP performance level indicate that this group only experience light damage. Moreover, since group 9 at the same level is selected for strain gage placement, this group can be also omitted from sensor installation plan.

The obtained usage ratios for groups 7 and 8 indicate that similar to groups 5 and 9, these two groups experience severe damage. The obtained results from nonlinear time history analysis show that, seismic-induced damage to these two groups concentrates on two distinct damage zones. The first damage zone is located between foundation level and 1.6 m high of the tower while the second damage zone is located between 4.0 m high and 8.2 m high of the tower. Therefore, these two zones are the best places for installation of strain gages. Based on the usage ratios obtained from CP performance level, groups 1 and 10 experience only moderate damage and they are able to maintain LS performance level. These two groups have negligible role in carrying gravity loads. Moreover, since the seismic behaviour of groups 5, 7, 8 and 9 that have the most contributions in carrying gravity loads are selected to be monitored; these two groups can be also omitted from the sensor installation plan. There is no need to install strain gage on elements of group 6 because the seismic behaviour of columns below this level (i.e., group 5) was already selected to be monitored. Furthermore, as Fig. 4 shows group 2 is subjected to severe damage, it should therefore be considered for strain gage installation. Considering abovementioned results, Table 6 compares the number of required strain gages based on a uniform distribution with those obtained from the proposed method. In this table for the sake of comparison, it was assumed that at every 5 m^2 of concrete walls and slabs one strain gage is needed to be installed. In addition, only one strain gage is considered for each vulnerable structural component. As this table shows, by using the proposed method the number of required strain gages is significantly decreased.

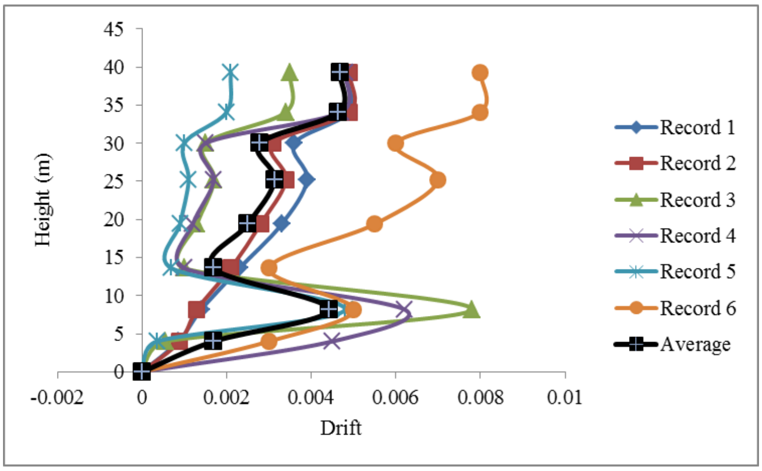

Fig. 4 Inter-story drift demands along the height of tower

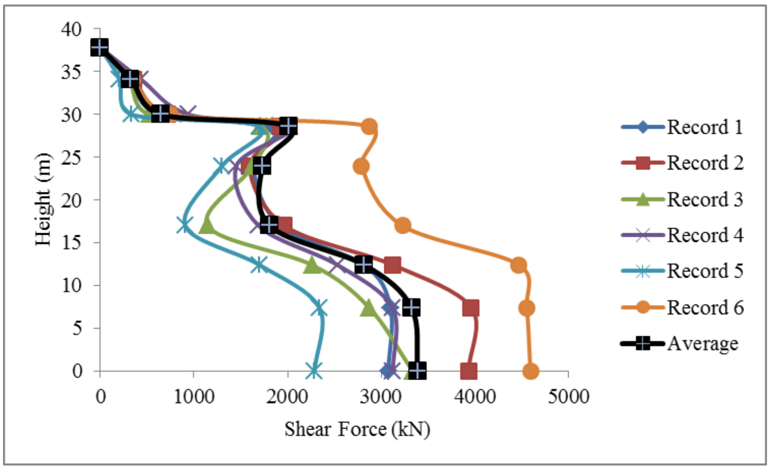

Fig. 5 Shear force demands along the height of the tower

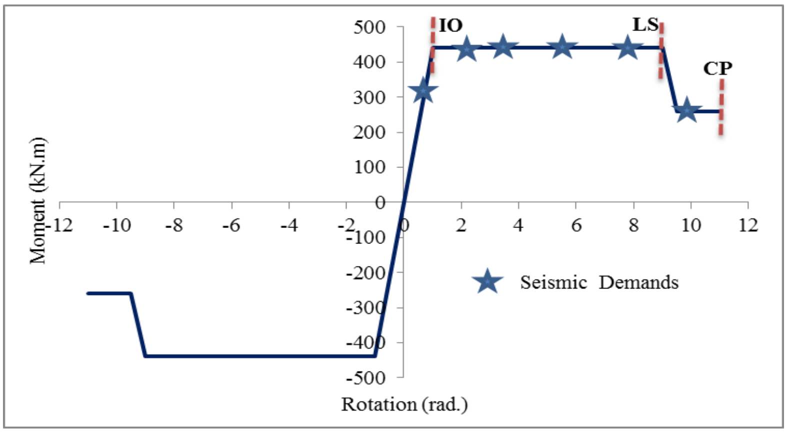

Fig. 6 Modeling parameters and acceptance criteria for beams in group 2

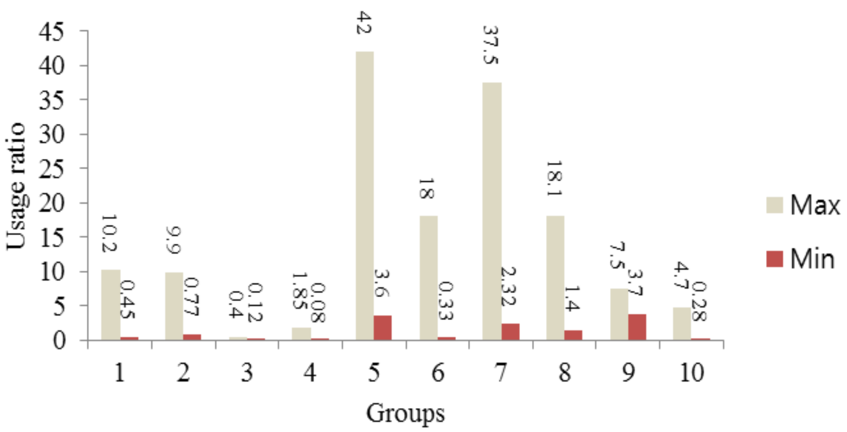

Fig. 7 Envelop of maximum and minimum usage ratios obtained from nonlinear time history analysis for IO performance level

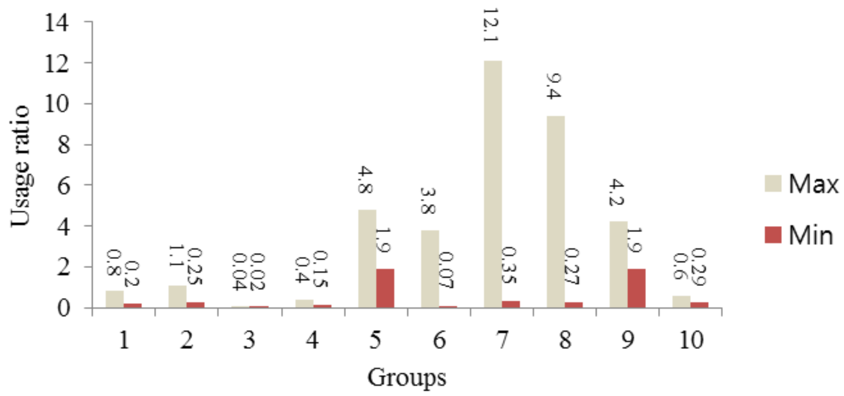

Fig. 8 Envelop of maximum and minimum usage ratios obtained from nonlinear time history analysis for CP performance level

Table 6 Number of required strain gages based on uniform distribution and the proposed method

Groups	Uniform distribution	Proposed method
Group 1	6	0
Group 2	12	12
Group 3	6	0
Group 4	12	0
Group 5	12	12
Group 6	6	0
Group 7	78	25
Group 8	48	12
Group 9	12	12
Group 10	14	0
Total	206	73

7. Conclusions

Optimal sensor placement is of great importance for structural health monitoring system. On one hand, optimal sensor placement directly reduces the cost of SHM systems because the number of installed sensor decreases. On the other hand, by decreasing the number of sensors, storage of uninformative data is significantly reduced and data processing becomes more efficient. In this study, a method for ideal strain gage placement was proposed. The proposed method is in line with the concept of performance based seismic design. At first, structural performance levels and seismic hazard levels are determined. Then by using nonlinear time history analysis the usage ratio of structural components are calculated. Finally, by comparing the obtained usage ratios the most vulnerable elements are identified for strain gage installation. The feasibility of the proposed method was demonstrated by testing it on the ATC tower of Kerman International Airport, Iran. The obtained results revealed that, the proposed method decreased significantly the number of required strain gages while maintaining the efficiency of the SHM system.

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