# Seismic Response of Pre-Cast Industrial Buildings During 1999 Kocaeli Earthquake

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ABSTRACT: Pre-cast frame structural system is widely accepted for the industrial structures throughout Turkiye, due to its rapid and economical construction practice. Field investigations after the 1999 Kocaeli and Duzce Earthquakes revealed that the damage in such structures was primarily due to the improper detailing in beam column connections and lack of required lateral stiffness. This paper presents a particular pre-cast frame system called *Lambda Frame* with emphasize on its seismic performance and damage distribution observed during the '99 earthquakes. Three *Lambda Frame* industrial structures with different damage levels were thoroughly investigated and analyzed with linear-elastic material modeling. A parametric study on changing column dimensions was performed with reference to the attained drift levels both under the code specified loads and under the '99 acceleration records. A model, which considers the dynamic properties of the pre-cast structure and the engineering characteristic values of the earthquake record, is proposed for the seismic performance of *Lambda Frame* structures.

KEYWORDS: Kocaeli earthquake, pre-cast frame structure, beam column connection, seismic performance model, pre-cast lambda frame

## 1 INTRODUCTION

In the recent years pre-cast concrete frame structures, which are fixed at the column base and with pinned beam-to-column connections at the roof level, are mostly favored for the single story industrial type of buildings due to its high construction speed and relatively low cost. Pinned connections in such frames may be constructed either on the column tops or at points of contra-flexure under gravity loads over the roof girders. Frames with pinned beam-to-column connections at locations of contra-flexure are called "Lambda Frames".

A widespread damage was observed in pre-cast industrial construction after the '99 Kocaeli and Düzce Earthquakes, owing to the industrial density in the East Marmara Region where Kocaeli and Düzce are located. Field investigations and analytical evaluations revealed that a high percentage of such structures didn't have satisfactory earthquake safety. The reason may be attributed to the ignorant transfer of design know-how from non-earthquake prone countries and lack of modification to make the structure have adequate lateral load capacity. Low degree of structural system indeterminacy, lack of diaphragm action at the roof level, inadequate lateral stiffness and insufficient connection strength and detailing may be listed as the main reasons for the damage in pre-cast industrial lambda frames of the region.

Within the scope of the study outlined here, field investigations to highlight the common failure types, structural deficiencies, and different damage levels of lambda frames was performed and three lambda frame buildings were selected for a thorough analysis with linear elastic material modeling. Natural period of vibration characteristics and roof level displacement values under inertial forces calculated according to the new Turkish Earthquake Code (ABYYHY-98) approach on as-built structural systems were evaluated.

A parametric study on changing column dimensions was performed with reference to the drift level of a sample lambda frame and a linear time history analysis using the ’99 acceleration records was completed in order to investigate the maximum drifts attained under the recorded ’99 earthquakes. A model for the seismic performance evaluation of *Lambda Frame* structures is proposed on the basis of the dynamic characteristic values of the industrial structure and the applied earthquake record.

## 2 EARTHQUAKE MOTION

Turkiye experienced two major earthquakes in 1999, which occurred 86 days apart on The North Anatolian Fault system (Figure 1). The fault rupture length was approximately 140 km ($M_w$=7.4) during the August 17, 1999 Kocaeli Earthquake, where the epicenter was located at 40.76$^o$N and 29.97$^o$E and the depth was 16 km. Three months after the August 17 event, $M_w$=7.2 Düzce earthquake ruptured another 40 km segment of the same fault to the east. The epicenter of the second earthquake was located at 40.76$^o$N and 31.15$^o$E and the focal depth was at 12 km (Sucuoğlu H., 2000).

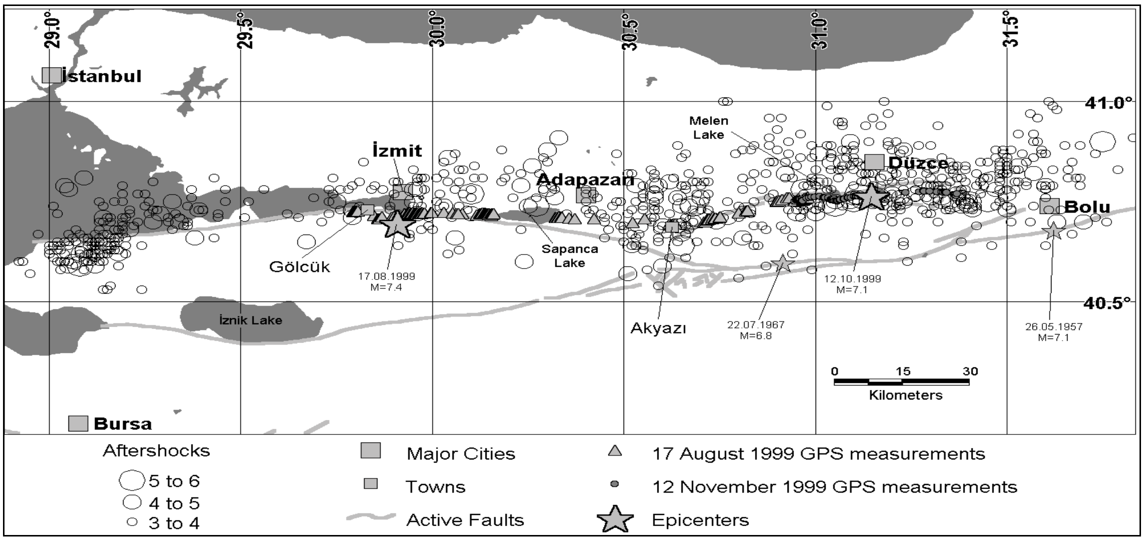


Figure 1. Kocaeli and Düzce Earthquakes, (Sucuoğlu H., 2000).

Various stations recorded the strong ground motion with engineering significance during Kocaeli and Düzce earthquakes, among which 17 of them were used in the linear elastic analysis of the sample lambda frames. Engineering properties of these earthquake records are given in Table 1.

## 3 EARTHQUAKE SAFETY

Structural system behavior coefficient for the pre-cast frames is given as $R$=5 in the Turkish Earthquake Code, and the design loads are $1/R^{th}$ of the elastic base shear. In determining the earthquake safety, inter-story drift or roof drift ratios in the case of single story frames under the design loads are compared with the upper limits given in the Code (Equations 1-a, and 1-b). Where, $\Delta_i$ is the relative lateral story displacement of the $i^{th}$ story, $h_i$ is the story height and $R$ is structural system behavior coefficient (ABYYHY, 1998).

$$(\Delta_i)_{\max} / h_i \leq 0.0035 \tag{1-a}$$

$$(\Delta_i)_{\max} / h_i \leq 0.02/R \tag{1-b}$$

Table 1. Engineering Properties of Earthquake Records from Kocaeli and Düzce Earthquakes

| Earthquake | Station | Component | Soil Condition | PGA (g) | PGV (m/s) | ARIAS Intensity | SMD (s) |
|---|---|---|---|---|---|---|---|
| Düzce | DZC | EW | Stiff Soil | 0,51 | 0,77 | 18,32 | 11 |
| | | NS | | 0,41 | 0,67 | 16,97 | 11 |
| | | Z | | 0,34 | 0,25 | 7,47 | 11 |
| Düzce | GYN | EW | Soft Rock | 0,02 | 0,05 | 0,11 | 23 |
| | | NS | | 0,03 | 0,08 | 0,13 | 21 |
| | | Z | | 0,02 | 0,06 | 0,10 | 20 |
| Düzce | IZN | EW | Stiff Soil | 0,02 | 0,04 | 0,03 | 15 |
| | | NS | | 0,02 | 0,04 | 0,05 | 12 |
| | | Z | | 0,01 | 0,07 | 0,01 | 19 |
| Düzce | IZT | EW | Rock | 0,02 | 0,05 | 0,03 | 11 |
| | | NS | | 0,02 | 0,06 | 0,03 | 11 |
| | | Z | | 0,02 | 0,06 | 0,02 | 11 |
| Düzce | MDR | EW | Rock | 0,06 | 0,20 | 0,58 | 18 |
| | | NS | | 0,12 | 0,17 | 1,28 | 16 |
| | | Z | | 0,06 | 0,15 | 0,48 | 19 |
| Düzce | BTS | EW | Stiff Soil | 0,00 | 0,01 | 0,00 | 25 |
| | | NS | | 0,00 | 0,02 | 0,01 | 27 |
| | | Z | | 0,00 | 0,01 | 0,00 | 27 |
| Düzce | HAS | EW | Soft Rock | 0,03 | 0,15 | 0,08 | 25 |
| | | NS | | 0,02 | 0,17 | 0,17 | 32 |
| | | Z | | 0,02 | 0,13 | 0,14 | 31 |
| Düzce | YPT | EW | Soft Rock | 0,02 | 0,04 | 0,08 | 44 |
| | | NS | | 0,02 | 0,09 | 0,11 | 40 |
| | | Z | | 0,01 | 0,04 | 0,04 | 51 |
| Kocaeli | CEK | EW | Stiff Soil | 0,09 | 0,15 | 0,75 | 9 |
| | | NS | | 0,12 | 0,12 | 1,16 | 10 |
| | | Z | | 0,05 | 0,06 | 0,28 | 10 |
| Kocaeli | DZC | EW | Stiff Soil | 0,37 | 0,52 | 8,45 | 12 |
| | | NS | | 0,32 | 0,60 | 6,82 | 12 |
| | | Z | | 0,48 | 0,21 | 3,38 | 12 |
| Kocaeli | ERG | EW | Unknown | 0,10 | 0,13 | 0,98 | 15 |
| | | NS | | 0,09 | 0,14 | 0,94 | 20 |
| | | Z | | 0,06 | 0,07 | 0,20 | 16 |
| Kocaeli | GBZ | NS | Rock | 0,27 | 0,61 | 3,69 | 30 |
| | | Z | | 0,20 | 0,22 | 1,29 | 30 |
| Kocaeli | GYN | EW | Soft Rock | 0,12 | 0,14 | 1,23 | 12 |
| | | NS | | 0,14 | 0,13 | 1,59 | 12 |
| | | Z | | 0,13 | 0,17 | 0,90 | 13 |
| Kocaeli | IZN | EW | Stiff Soil | 0,12 | 0,30 | 3,23 | 33 |
| | | NS | | 0,09 | 0,18 | 1,86 | 34 |
| | | Z | | 0,08 | 0,10 | 0,83 | 34 |
| Kocaeli | IZT | EW | Rock | 0,23 | 0,52 | 6,02 | 35 |
| | | NS | | 0,17 | 0,42 | 4,53 | 34 |
| | | Z | | 0,15 | 0,48 | 2,62 | 34 |
| Kocaeli | HAS | EW | Soft Rock | 0,11 | 0,92 | 1,73 | 44 |
| | | NS | | 0,05 | 0,74 | 0,60 | 63 |
| | | Z | | 0,14 | 0,35 | 1,36 | 38 |
| Kocaeli | YPT | EW | Soft Soil | 0,23 | 0,86 | 9,90 | 33 |
| | | NS | | 0,32 | 0,85 | 10,08 | 32 |

Performance level of building structures may be defined as the attained level of damage after a big earthquake where the damage level may vary between no damage and total/partial collapse. Structural system performance levels defined in FEMA-273 are given in Table 2, with reference to the inter-story drifts observed during an earthquake. Structure may be immediately occupied in the case of "S1" performance level, while the stiffness and the lateral load carrying capacity of the structure is assumed to be very limited and any kind of occupancy is prohibited for the "S-5" performance level (FEMA-273, 1997).

Table 2. Structural performance levels and displacements limits (FEMA-273, 1997)

| Structural Performance Level | Collapse Prevention S-5 | Life Safety S-3 | Immediate Occupancy S-1 |
|---|---|---|---|
| Drift | 4% transient or permanent | 2% transient; 1% permanent | 1% transient; negligible permanent |

Overwhelming majority of the columns in pre-cast industrial structures in the earthquake hit region were 50 by 35 cm or smaller and a parametric study conducted by Posada (Posada M., 2002) revealed that the connection detail and the column dimensions have direct influence on the structural performance of pin connected pre-cast industrial buildings. Not only the structural dynamic properties, but also the engineering properties of the earthquake records have influence on the performance level of the structures. Observation of building structural and nonstructural damage after severe earthquakes, as well as numerical investigations of these structural systems, exhibit a more or less marked interdependency between structural indices and several acceleration parameters such as peak ground acceleration (*PGA*), peak ground velocity (*PGV*), peak ground displacement (*PGD*), the total duration of the event, *ARIAS* intensity, strong motion duration (*SMD*), spectral pseudo-acceleration (*SA*), spectral pseudo-velocity (*SV*), and spectral pseudo-displacement (*SD*) (Elenas A., 2000).

## 4 STRUCTURAL PROPERTIES OF INDUSTRIAL BUILDINGS INVESTIGATED

Many pre-cast industrial buildings were investigated in the earthquake-hit region after the Kocaeli and Düzce earthquakes. Pre-cast lambda frame structures were prevalently used in one-story large space industrial buildings. These buildings tend to be rectangular in plan with one to four bays in transverse direction and ten to thirty bays in the longitudinal direction. Transverse bay widths ranged from 10 to 25 m and longitudinal bay widths ranged from 6 to 8 m. Column heights also ranged from 6 to 8 m (Figure 2). Pre-cast columns were grouted into single cast in place or pre-cast footings and the footings were cross-tied with grave beams. The connection between the individual lambda frames was with gutter beams and purlins that were pinned at both ends.

The connecting members (columns and beams) in the lambda frames were tapered over a certain length, and pinned with one or two bolts passing through both of the members (Figure 2). In the parametric analysis of the sample lambda frame and the selected as-built pre-cast industrial structures with lambda frames, flexural stiffness of the connection region was decreased approximately by %70, with respect to the connecting member stiffness values (Arslan H., 2000). On the other hand, the gutter beam and purlin connections were true pin connections with no moment transfer capability. Roof cover of these industrial buildings was either 3-4 mm cement based corrugated composite sheets or aluminum panels with heat insulation in between.

Field observations on pre-cast lambda frames revealed that most of such buildings underwent large lateral displacements due to small column dimensions, as a result the connections were extremely stressed and the connecting bolts at the beam column connections either slipped through the holes or simply failed under tension. In a certain number of structures, deformed reinforcing bars with cement grouting were used in the connection holes as dowels. In such connections, roof beams were simply fall apart from the connection leading to a total collapse of the roof cover. No diaphragm action, which leads to a force re-distribution among the lateral load carrying system, was observed in the pre-cast lambda frame industrial buildings (Ersoy et.al., 2000).

After the '99 Kocaeli and Düzce earthquakes three lambda frame structures, namely EM, KM and SB, were selected for further investigation. The structure EM collapsed during the Kocaeli earthquake, while structure KM experienced moderate damage and SB showed minor structural damage. All the buildings were constructed on similar soft soils and located at a distance of approximately 30 km with respect to each other. Building dimensions are given in Table 3.

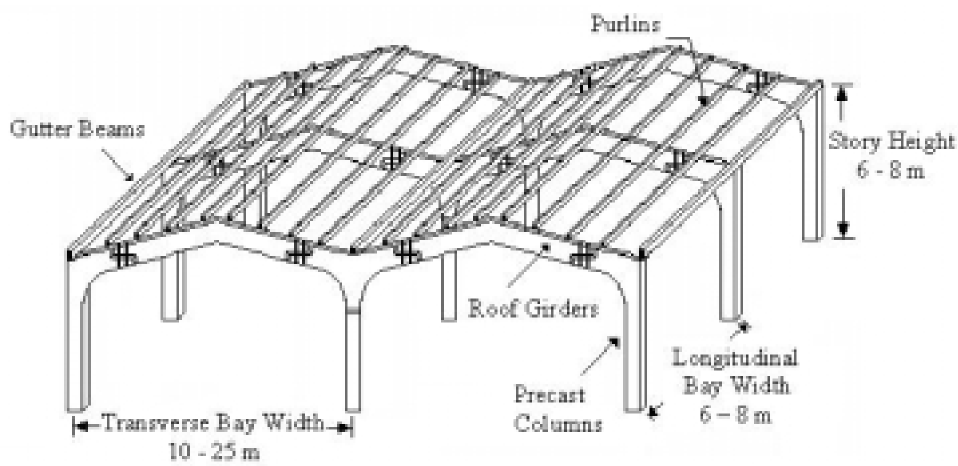
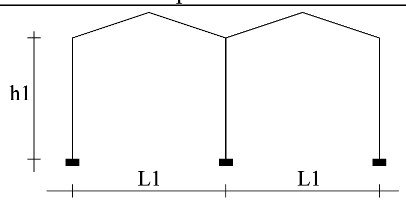
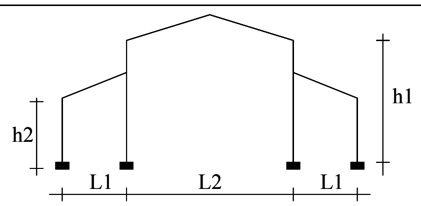



Figure 2. Typical Configuration of Lambda Frame Industrial Buildings.

Table 3. Dimensions of pre-cast industrial buildings (in meters)




(A)




(B)

| Building Designation | EM | KM | SB |
|---|---|---|---|
| Lambda Frame Type | (A) | (B) | (A) |
| Number of Bays in Transverse Direction | 2 | 3 | 2 |
| Number of Bays in Longitudinal Direction | 9 | 9 | 8 |
| L1 | 20 | 11 | 21 |
| L2 | - | 20 | - |
| h1 | 7.5 | 9.0 | 6.5 |
| h2 | - | 5.0 | - |
| Column Dimensions (bldg. longitudinal x transverse direction) | 0.25 x 0.50 | 0.30 x 0.55 | 0.30 x 0.55 |
| Roof Beam Dimension (width x depth) | 0.25 x 0.50 | 0.30 x 0.50 | 0.30 x 0.50 |

The selected structures, namely EM, KM and SB, were analyzed to determine the dynamic properties and the maximum roof level drift ratios under the inertial loads calculated according to the new Turkish Earthquake Code (ABYYHY, 1998) and the results are given in Table 4.

Table 4. Response Spectrum Analysis Results of Sample Buildings

| Building | E.Q. Direction | Natural Period (sec) | Drift Ratio (Δ/h) |
|---|---|---|---|
| EM | Transverse | 0.73 | 0.0051 |
| | Longitudinal | 1.99 | 0.0126 |
| KM | Transverse | 0.36 | 0.0014 |
| | Longitudinal | 1.57 | 0.0054 |
| SB | Transverse | 0.59 | 0.0047 |
| | Longitudinal | 0.75 | 0.0001 |

## 5 EFFECT OF COLUMN STIFFNESS ON STRUCTURAL PERFORMANCE

A model Lambda Frame with two bays in the transverse direction was parametrically studied. The transverse bay width was 20 m each and the height was 7.5 m. Columns were fixed at the base, and the roof girder to column connection flexural stiffness was assumed to be 30 percent of the connecting beam stiffness. Linear elastic material modeling with a concrete Modulus of Elasticity of $Ec$=34 GPa was used in the structural modeling. The mass was distributed over the length of the bar elements for the Lambda Frame, while point mass approach was used for the mass of gutter beams, purlins and the roof cover. Longitudinal bay width was taken as 7.5 m in calculating the point mass values. Primarily, the effect of column dimension on the natural period of vibration was investigated and the results are listed in Table 5.

Table 5. Effect of Column Size on Natural Period of Vibration

| Column Dimension | | Calculated Natural Period | |
|---|---|---|---|
| Transverse (m) | Longitudinal (m) | Transverse (second) | Longitudinal (second) |
| 0.50 | 0.25 | 0.71 | 2.23 |
| 0.50 | 0.35 | 0.64 | 1.51 |
| 0.50 | 0.45 | 0.59 | 1.14 |
| 0.50 | 0.50 | 0.57 | 1.03 |
| 0.55 | 0.55 | 0.50 | 0.93 |
| 0.55 | 0.60 | 0.48 | 0.88 |
| 0.60 | 0.60 | 0.44 | 0.86 |
| 0.60 | 0.65 | 0.43 | 0.82 |
| 0.65 | 0.65 | 0.39 | 0.81 |
| 0.70 | 0.70 | 0.35 | 0.78 |
| 0.75 | 0.75 | 0.31 | 0.75 |
| 0.80 | 0.80 | 0.28 | 0.74 |
| 0.85 | 0.85 | 0.26 | 0.72 |
| 0.90 | 0.90 | 0.23 | 0.71 |
| 1.00 | 1.00 | 0.20 | 0.70 |

Time-history analysis with acceleration records summarized in Table 1 was also performed on the model lambda frames with different column dimensions listed in Table 5. Relation between the fundamental period of vibration and the maximum drifts attained was sought under the effect of accelerations recorded during the ’99 Kocaeli and Düzce earthquakes. It is assumed that the same relation is valid for the 3-D Lambda Frame structures, namely EM, KM and SB.

The maximum drifts in the longitudinal direction for the model Lambda Frames analyzed with 17 different acceleration records are plotted against the fundamental period of vibration of the frame and shown in Figure 3. FEMA-273 drift limits were used for the performance level evaluation of the results. It is observed that the frames with a fundamental period of vibration smaller than 0.7-0.8 sec. may be classified in the immediate occupancy performance level, while it is in collapse prevention performance level in case the natural period ranges between 1.2 and 1.5 sec. Structures with fundamental periods between 0.8 and 1.2 sec. satisfied the life safety performance level.

## 6 INFLUENCE OF EQ PARAMETERS ON STRUCTURAL PERFORMANCE

The damage level of a structure may be expressed as the sum of the damage accumulated on individual members. It is observed that the damage level of structural members may be interrelated to the maximum plastic chord rotation attained during the seismic excitations. The available parameters in the literature that quantify the structural damage level may be called as damage indices (Elenas A., 2001). Since the damage index calculation uses inelastic material modeling and time history analysis, such procedures need detailed knowledge of the member properties and reinforcement detailing. Hence the damage index calculations are limited in use.

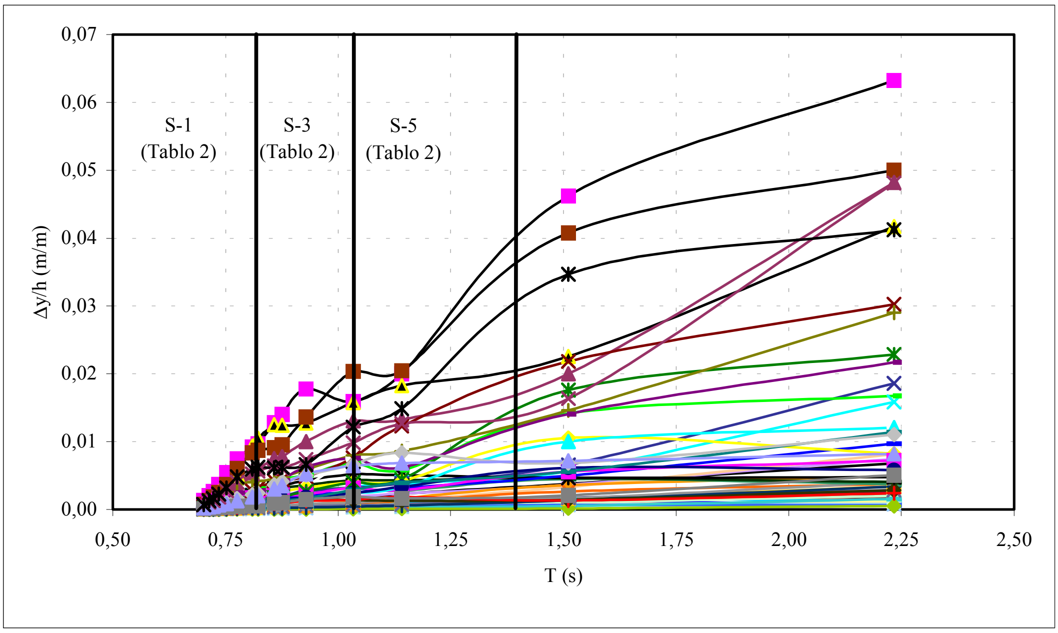



Figure 3. Natural Period versus Lateral Drift in Longitudinal Direction

Structural damage index values are closely related to the structural properties of the buildings, as well as the engineering properties of the acceleration records such as the peak ground acceleration, velocity, displacement (*PGA, PGV, PGD*) and the spectral pseudo acceleration, velocity, displacement (*SA, SV, SD*) values of the earthquake record. The energy intensity (*ARIAS*) (Equation 2) and the duration of the event also influence the level of the overall damage. Analytical models with non-linear material modeling revealed that the damage index values are closely related to the *SA*, while medium correlation exists between the damage index and the *PGV*, *PGD*, and *ARIAS* values (Elenas A., 2001).

$$I_0 = \int_0^{t_e} (\ddot{u})^2 \, dt \qquad (2)$$

In the last part of the study outlined here, a structural performance model for the Lambda Frame systems is proposed. Distinctive characteristic of the structure that is the fundamental period of vibration was interrelated to the damage level, and the properties of the forcing function, which was the acceleration record, was modeled by using the ARIAS and SA values. Parametric study results were evaluated such that the maximum roof drift was classified according to FEMA-273. The results of the parametric study were plotted in Figure 4. The damage level for a particular frame that is given in Table 5 analyzed under a certain earthquake record of Table 1 was named as Low Damage (*LD*) in case the maximum roof level drift attained was smaller than 1%, while 2% drift limit was assumed as the upper limit of the Medium Damage (*MD*), and the lower limit for the High Damage (*HD*) performance levels.

As shown in Figure 4, the calculated damage level increases as either the fundamental period of vibration of the Lambda Frame or the (*ARIAS / SA*) ratio of the acceleration record increases. Roof drift values, which were used as the indication of the damage level, were smaller for stiff structures no matter which earthquake record was used. It was observed from Figure 4 that points showing the damage levels grouped in three regions, hence boundary limits were drawn accordingly. The buildings EM, KM, and SB were used to calibrate the proposed linear elastic performance model for the pre-cast lambda frame structures. The (*ARIAS / SA*) value of the YPT record (Table 1) was used for these buildings. It should be noted that, EM was totally collapsed during the earthquake, while KM had repairable damage and SB was operational right after the event.

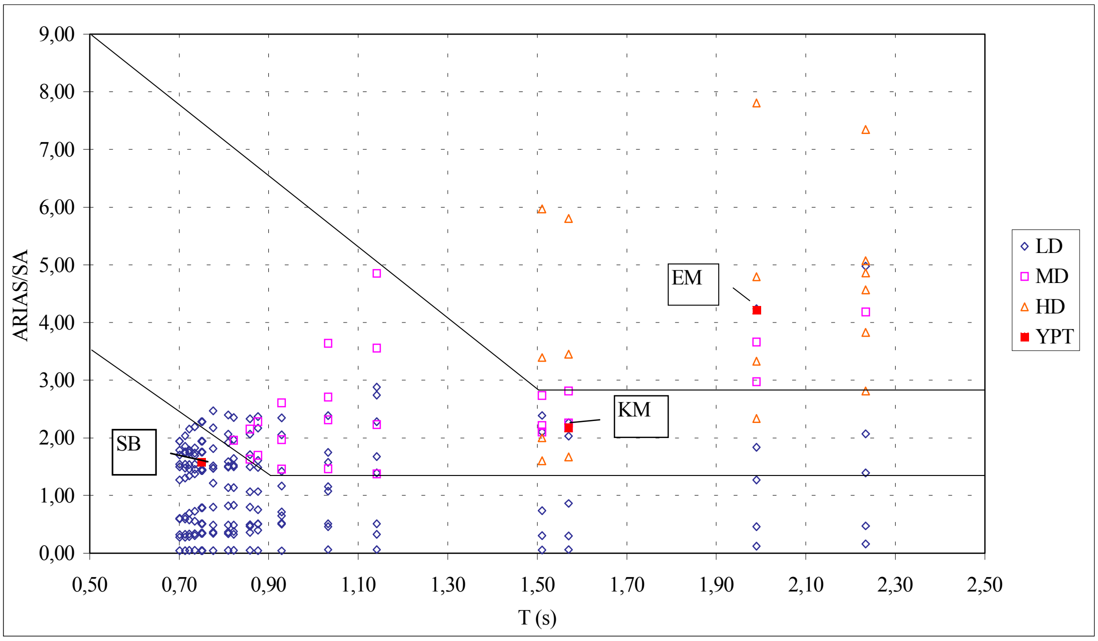


Figure 4. Damage Level Model for Lambda Frame Industrial Structures

## 7 CONCLUSIONS

The damage accumulated in the pre-cast concrete Lambda Frame industrial structures during the '99 Kocaeli and Düzce earthquakes may be attributed to the deficiencies in the beam column connection regions as well as the inadequate lateral stiffness of such frames. It was observed that the performance level of Lambda Frame structures may be predicted from a model interrelating the fundamental period of vibration of the structure and some engineering properties of the earthquake excitation. Although the outcomes of the proposed model and the actual observations on the field are in good correlation, more field data is needed for further calibration of the model

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# Seismic Risk Assessment and Mitigation Measure Program in Algeria

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As Algeria is subjected to earthquake due principally to the seismotectonic setting of the Maghreb region, the authorities have devoted particular efforts for seismic risk prevention by creating special organization dealing with seismology and earthquake engineering, installing seismic observation systems and elaborating a code of earthquake resistant design of structure in order to reduce the consequences of this natural disaster. Hence, many organizations have been created and many professionals have been trained.

KEYWORDS: Earthquake, Seismic risk, Seismic hazard, Disaster.

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KEYWORDS: Kocaeli earthquake, Pre-cast frame structure, Beam column connection, Seismic performance model, Pre-cast lambda frame.

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# ABSTRACTS

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