PERFORMANCE OF SCHOOL CANOPIES DURING THE 10TH OCTOBER 1980 EL-ASNAM, ALGERIA EARTHQUAKE

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SUMMARY

The performance of canopies in a school near El-Asnam, Algeria, during the earthquake of October 10, 1980, is studied primarily to determine the peak effective acceleration, *EPA*, of the earthquake capable of inducing the observed damage in the city of El-Asnam (Ref. 1). Although a small range of values for *EPA* (i.e., 0.28g to 0.35g) is determined for the two ground motions considered, many uncertainties exist in the determination and use of *EPA*. As the sole parameter by which the damaging potential of an earthquake is judged, the factor *EPA* can be extremely misleading.

INTRODUCTION

On October 10, 1980, a destructive earthquake of Richter magnitude (M) 7.2 occurred near El-Asnam, Algeria. The epicenter was about 10 km (6 mi) east of El-Asnam, with a focal depth of 10 km. The duration of the earthquake was between 35 and 40 seconds. No strong motion records of the main shock were obtained. Field estimates, however, place the peak ground acceleration at more than 0.40g (Ref. 2).

Although the effective peak acceleration, *EPA*, is a quantity theoretically associated with the damaging potential of a ground motion, the parameter has yet to be systematically defined. Blume (Ref. 3) defines *EPA* as "the acceleration which is fully effective in causing structures to respond, whereas any acceleration with a greater value is not effective at all." A detailed analysis of several canopies in a school located 5 km east of El-Asnam was performed in an attempt to ascertain first whether the value *EPA* could be determined and second whether that value represented the damaging potential of a ground motion.

In the series of nonlinear dynamic analyses conducted on these canopies, certain parameters for which no reliable information was available were varied. Due to a lack of data on the mechanical characteristics of the materials used and the sensitivity of the dynamic response of the building to the dynamic characteristics of the ground motion, the results described here should be viewed as crude estimates of *EPA*.

ANALYTICAL MODELING OF THE STRUCTURAL SYSTEM

The structural system of the school consisted of a row of four cylindrical columns supporting a very massive slab (3.50 x 19.45 meters in plan) at the top (Fig. 1). The canopies covered the corridors along and between the classrooms and were completely free of nonstructural components. The seismic response of the canopies thus depended solely on the bare simple structural system.

The structural failure of the canopies was triggered by the failure of the columns at their base in the transverse direction. This failure was, in turn, triggered by flexural yielding of the main reinforcement followed by crushing of the concrete and buckling of the longitudinal compressive steel bars. Some minor damage occurred throughout the columns, especially at the top. The roof of the canopy sustained no damage and the foundation apparently did not move during the shaking, i.e. the foundation behaved as a rigid system. The structural system is illustrated in Fig. 1 and is mechanically idealized as shown in Fig. 2, where: L equals 2.67 m; M (concentrated mass at the roof) equals 1.17 ton-sec^2/m; $\overline{m}$ (distributed mass of the columns) equals 0.0023 (ton-sec^2/m)/m; I_0 (mass moment of inertia of roof) equals 1.23 ton-sec^2-m; EI_{uc} (flexural stiffness of uncracked, transformed cross section of column) equals 1843 ton-m^2; and EI_{cr} (cracked transformed cross section of the column) equals 769 ton-m^2.

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While the degrees of freedom of this idealized system are infinite, it can, due to the relatively small value of the mass distributed throughout the column ($\overline{m}$) compared with the mass concentrated at the top of the canopy (M), be adequately represented by an equivalent two-degree-of-freedom system (2D0F), ν_1 and ν_2, in which the translational and rotational masses are considered to be concentrated at the top of the column of the canopy (Fig. 2). Studies were also conducted in which a single degree of freedom (SDOF) was considered, i.e. ν_1 alone.

ESTIMATION OF MECHANICAL (DYNAMIC) CHARACTERISTICS

Mechanical Characteristics of Structural Materials

The canopies were constructed of reinforced concrete. No testing data or design values used for the concrete or the reinforcing steel strengths were available. Based on visual observations of these materials and on conversations with those involved in the construction, the following probable mechanical characteristics were assumed.

1) **Concrete Stress-Strain Relationships.** The following upper and lower bound values for 28-day cylinder strength of the concrete, f_c', were assumed. Case 1: $f_c' = 2100\ t/m^2$; Case 2: $f_c' = 1750\ t/m^2$. The Park-Kent concrete stress-strain curves were used to derive the confined and unconfined curves for both types of concrete (Refs. 4 & 5).

2) **Reinforcing Steel Stress-Strain Relationships.** The following upper and lower bounds for the stress at yield, f_y, were assumed: Case 1: $f_y = 35000\ t/m^2$; Case 2: $f_y = 28000\ t/m^2$.

Although analyses were performed for all combinations of the values given above, the predicted values reported here are those for Case 2, considered to be the more realistic.

Computed and Idealized Moment-Curvature (M-Φ) Relationships of Column Cross Sections

Numerical computations of the M-Φ relationships were conducted using the RCCOLA computer program (Ref. 6), in which the variations of mechanical characteristics stated above and the effect of the following parameters were considered:

1) **Axial Load P.** Since the weight tributary to each of the columns of the canopy was approximately 12 tons, and since the vertical component of the ground motion was estimated to be as high as 1.0g (Ref. 2), the M-Φ curves were calculated for axial loads of 0, 12, and 24 tons. From these curves it was concluded that the effect of axial force on the M-Φ relation could be neglected. A value for P of 12 tons was assumed in subsequent analyses.

2) **Cover Thickness.** The effect of two cover thicknesses, $d' = 0.043$ and 0.056m (where d' is the distance from the center of the longitudinal steel reinforcement to the outside edge of the concrete on the same side of the section), on the behavior of the column was investigated. The main effect of an increase in cover is a decrease in moment yielding and maximum flexural strength of less than 10% (Fig. 3). A value of 0.056m was subsequently used for d'.

The moment rotation capacity rather than the moment-curvature relation of critical regions of reinforced concrete structures is important in determining seismic response. Because the spacing of the circular hoops (0.18m) was larger than the maximum 0.10m permitted by the 1982 UBC [Ref. 7] for seismic zones 3 and 4, and even larger than the eight-bar diameter allowed for buckling (0.15m), the possibility of premature buckling after the confined concrete of the cover shell had spalled had to be considered. In the few canopy columns that remained standing, there was evidence that the bars may just have begun to buckle. The moment-rotation relation, M-Θ, along a region of length l along which the bar could buckle was determined assuming that the moment and curvature along the critical region were constant: $\Theta = \Phi l$. The M-Φ at which buckling would be initiated is illustrated in Fig. 3. The curvature at which buckling would begin was not uniquely determined. Instead, a range of values was obtained which bounded the uncertainty: somewhere between the start of spalling of the concrete shell and the initiation of strain hardening of the steel. However, it was believed that buckling would initially occur at a point closer to the first value: spalling of the concrete shell.

The M-Φ relation was idealized in order that it might be used as input data for the DRAIN-2D computer program (Ref. 8). The elastic/perfectly plastic idealization seemed reasonable for the range of curvatures investigated. No increase in moment due to strain hardening was considered since buckling of the longitudinal compressive steel bars occurred before then. For the idealized M-Φ curve, the linear elastic stiffness, EI, was estimated to be 769 $t-m^2$ and the maximum moment was 9.24 $t-m$.

Shear Strength of Column

The resistance to shear, V, depends on the shear strength provided by the concrete, V_c, and by the shear reinforcement, V_s. When these two quantities were considered, the available shear strength was found to be considerably higher than the shear demand from the lateral force required to induce flexural failure. However, the 1982 UBC specification, 2626(f)5, requires that the contribution of concrete be neglected when $P_e/A_g < 0.12f'_c$. The columns considered here should thus fail in shear even before yielding begins. Damage in the standing canopies and the type of failure observed in the collapsed canopies revealed that the failure was due to flexural yielding and not to shear. Therefore, although according to present UBC recommendations the shear strength should have controlled the failure, shear was not considered to be a problem.

Damping Ratios and Periods of the Canopy

A damping ratio, ξ, of 2% for the first mode of vibration was chosen since only a simple bare R/C structural system was vibrated. Studies in which ξ was set equal to 5% were also conducted.

When a SDOF system was assumed, the period of the structure was computed using Rayleigh's Method (Ref. 9), and assuming a first mode shape. When the 2DOF system (Fig. 2) was considered, the analyses yielded values for the first mode similar to that computed for the equivalent SDOF system. For the 2DOF system, the following values of T were estimated: 1) $T_1 = 0.46$ sec. and $T_2 = 0.11$ sec. when the column stiffness was computed on the basis of the uncracked, transformed section, EI_{uc}; and 2) $T_1 = 0.72$ sec. and $T_2 = 0.18$ sec. when the column stiffness was computed on the basis of the cracked, transformed section, EI_{cr}. Since the canopy was assumed not to have been cracked significantly prior to the earthquake, a value of T equal to 0.50 sec. was used to estimate the peak ground acceleration (PGA).

ANALYSES TO ESTIMATE THE EFFECTIVE PEAK ACCELERATION (EPA)

Introductory Remarks

Generally, EPA is less than the peak recorded acceleration, PGA, because acceleration pulses of high value but short duration have little effect on the response of most structures, and also because for structures strained into the inelastic range, the number and order of the acceleration peaks, as well as the duration of these pulses and the ground motion, rather than the peak itself, greatly influence damage (Ref. 10).

Besides the estimation of period, damping, and structural mass, as discussed above, the following problems were encountered in estimating EPA:

1) **Type of Ground Motion.** This is probably the main uncertainty in determining EPA. In this study, two types of earthquake were considered to represent the demands of inelastic deformation: the El Centro 1940 S00E and Derived Pacoima Dam 1971 S16E earthquake records. For the demands of inelastic deformation and for structures with $T >$ 0.4sec., the El Centro record may be considered a lower bound. Its major effect is thus to amplify the response of these structures by a resonance phenomenon. The Derived Pacoima Dam record is of the impulsive type (intense and long-duration acceleration pulses) and is considered an upper bound on inelastic deformation.

2) **Idealization of Lateral Resistance Function**: **R** vs. v_1. Estimated M-Φ diagrams (Fig. 3) were used to predict the idealized linear elastic/perfectly plastic lateral resistance functions, R versus v_1, of the canopies (Fig. 4). The range of values of maximum displacement v_u used for these estimates was controlled by the buckling of the longitudinal compressive steel (0.088m - 0.160m). Buckling was assumed to occur approximately at v_u = 0.10m.

3) **Displacement Ductility (μ_δ) Determination.** a) *Modified Park and Paulay Approach* (Ref. 5). These authors offer an approximate solution for the relationship between curvature ductility and displacement ductility ratio for a cantilever column with a lateral load at the end. The solution involves an assumed curvature distribution and an estimated length of plastic region, L_P. This approach was modified to include the influence of the top moment on the displacement ductility. Considering the initiation of buckling to be when spalling of the concrete cover occurred, Φ_u = 0.0898 rad/m and a value of μ_δ = 5.0 was determined. B) *Alternative Approach.* The canopies underwent a lateral displacement of at least 0.15m, the distance that separated the canopies and the adjacent classrooms, without failure. With this displacement assumed as an upper bound on v_u and using v_y = 0.029m (Fig. 4), μ_δ = 5 was estimated.

Estimation of EPA

Although different approaches can be used to estimate *EPA*, the approach presented in this paper involves three stages:

First Stage: Procedure Proposed by V. V. Bertero, S. A. Mahin, and R. A. Herrera. The maximum *PGA* is estimated from the charts developed by Bertero *et al.* (Ref. 11). For a given or selected ground motion, it is necessary to determine first the coefficient η which depends on the known or estimated values of period, damping, and displacement ductility (Fig. 5). The value of *PGA* is then computed directly from the following equation:

$$\eta = \frac{C_y}{|PGA|/g} = \frac{R_y}{m|PGA|}$$

For calculated values of m = 1.20 tons-sec^2/m, R_y = 3.46 tons, C_y = 0.29, and T = 0.50 sec, values of *PGA* equal to 0.55g and 0.35g for the El Centro and Derived Pacoima Dam records, respectively, were obtained.

Second Stage: Verification of PGA by DRAIN-2D. In the second stage, an inelastic dynamic computer program, such as DRAIN-2D (Ref. 8), is used to perform a series of time-history analyses for the earthquake motions normalized to values of *PGA* close to those values found in the first stage of the study. Both SDOF and 2DOF models were used.

For the SDOF model and *EI* approximately equal to that for a cracked section, a maximum displacement of approximately 0.10m was obtained for both the El Centro motion normalized to 0.55g and the Derived Pacoima Dam motion normalized to 0.35g (see Table 1). For the 2DOF model, the Derived Pacoima Dam motion of 0.35g also yields this value, but the El Centro 0.55g motion yields only 0.07m. These results indicate that the Bertero *et al.* charts yield good results for SDOF models, but that they do not work well for 2DOF systems, particularly for earthquake motions such as the El Centro motion. They also indicate that a 2DOF model is needed for further inelastic dynamic analyses.

TABLE 1 MAXIMUM LATERAL DISPLACEMENT, v_u

TYPE OF EARTHQUAKE	MODEL	EI ($t-m^2$) (cracked)	v_u (m)
El Centro 1940 (normalized to 0.55g)	SDOF	769	0.106
	2 DOF	769	0.070
Derived Pacoima Dam (normalized to 0.35g)	SDOF	769	0.105
	2 DOF	769	0.100

The results of the analyses show that to attain a v_u of 0.10m, the *PGA* should be close to 0.35g for the normalized Derived Pacoima Dam record and 0.65g for the normalized El Centro 1940 record.

Third Stage: Estimation of EPA by the "Clipping" Method. In the third stage, DRAIN-2D was used to carry out analyses assuming the two earthquake ground motions discussed above, but clipping the peak accelerations of these motions in order to study the influence that such "clipping" has on the response of the canopies.

By analyzing variations in the time-histories of the ground motions and the displacement response, the time at which the maximum displacement occurred can be determined and the peak of ground motion at which this displacement was produced can be identified. It is then possible to estimate how much the ground acceleration record can be clipped in order not to decrease the maximum response of the canopy.

The maximum top displacement of the canopy does not necessarily occur when the unclipped ground motion is used as input. When the peak ground accelerations that are clipped occur before the pulses inducing the maximum v_u, the clipping operation might modify the response of the structure in such a way that the particular values of the response at the time that the critical pulses begin favor the increase of the displacement due to these pulses. The acceleration clipping should be continued until the maximum displacement begins to decrease. The clipped (or unclipped) acceleration at which the maximum displacement occurred is considered the *EPA* for the case under consideration and for the specified earthquake. The effects of clipping are illustrated in Fig. 6 and in Table 2 the effect of clipping on the maximum lateral displacement response is summarized.

TABLE 2 EFFECT OF CLIPPING EL CENTRO NORMALIZED TO 0.55g & DERIVED PACOIMA DAM NORMALIZED TO 0.35g ON DISPLACEMENT RESPONSE OF THE CANOPY

DESCRIPTION OF EARTHQUAKE	PEAK CLIPPED ACCELERATION	MAXIMUM v_u	
		TIME (sec)	v_u (m)
EL Centro 1940 normalized to 0.55g	0.55g	2.01	0.070
	0.32g	5.38	0.098
	0.28g	5.38	0.104
	0.22g	5.37	0.101
Derived Pacoima Dam normalized to 0.35g	0.35g	3.65	0.100
	0.30g	3.65	0.089

For the Derived Pacoima Dam record normalized to 0.35g, any clipping of the ground acceleration decreases the lateral displacement response. Therefore, the *EPA* is 0.35g. The lateral displacement response increases, however, when the El Centro record is clipped, in this case from 0.070m (for the unclipped record normalized to 0.55g) to 0.104m when the record normalized to 0.55g is clipped to 0.28g. Since this value of v_u is close to 0.10m, the value sought, it was not necessary to increase the normalization of the El Centro record to, say, 0.65g as had been predicted in the second stage of the analysis.

In summary, for the El Centro record normalized to 0.55g, a value of 0.28g was estimated for the *EPA*. For the Derived Pacoima Dam record normalized to 0.35g, the predicted *EPA* equaled 0.35g.

SUMMARY AND CONCLUSIONS

1) A 2DOF model, rather than a SDOF system, must be considered.

2) Among the parameters that affect *EPA* are: a) the idealization of the lateral resistance function (which in turn depends on assumed material properties, concrete cover, axial load, buckling of longitudinal compressive steel bars, and fixed-end rotation, among other factors); b) model idealization (SDOF or 2DOF); c) period and damping, and d) ground motion characteristics.

3) Because the assumed recorded ground motions did not produce the observed response, these ground motions must be normalized to values of *PGA* as determined in the first two stages of the investigation. The values of *EPA* determined in the present investigation (i.e., 0.28g and 0.35g for the El Centro and Derived Pacoima Dam earthquake motions) were therefore derived from clipping distorted records, due to the normalization of *PGA* to 0.55g and 0.35g. In other words, the originally recorded El Centro record unnormalized with *PGA* equal to 0.35g produces low values of canopy displacement and therefore would not have resulted in

the observed damage, i.e. collapse of the canopy. Also, depending on the stiffness *EI* used in the analysis (and thus on the period), the *EPA* was or was not equal to the *PGA*. The amount of clipping therefore depends on the normalization of an earthquake.

4) The results obtained indicate that for the two assumed types of ground motion, the *EPA* needed to induce the observed damage is in the range 0.28g to 0.35g. This small range is apparently fortuitous given the wide range of the parameters that affect the result. Generally, *EPA* depends both on the type of earthquake considered and on the interaction of the dynamic ground motion characteristics and the soil-foundation-structure system. Furthermore, *EPA* will depend on the limit state under consideration. Although the use of *EPA* can provide an idea of the relative damaging potential of a given ground motion, its use as the sole parameter to define this damaging potential can be very misleading.

ACKNOWLEDGMENTS

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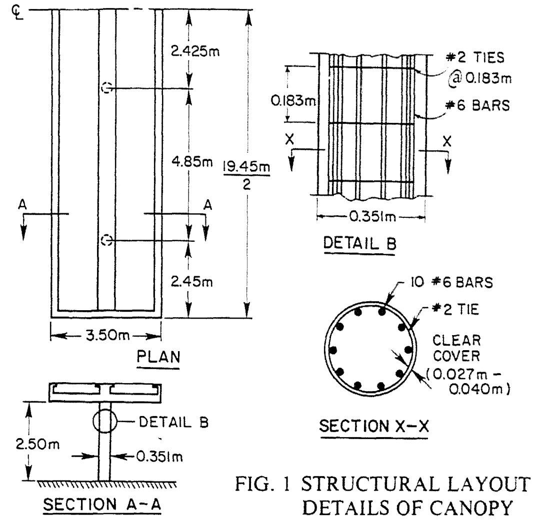


FIG. 1 STRUCTURAL LAYOUT & DETAILS OF CANOPY

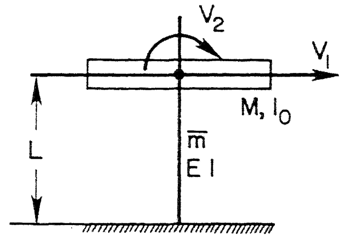


FIG. 2 ANALYTICAL MODEL OF CANOPY

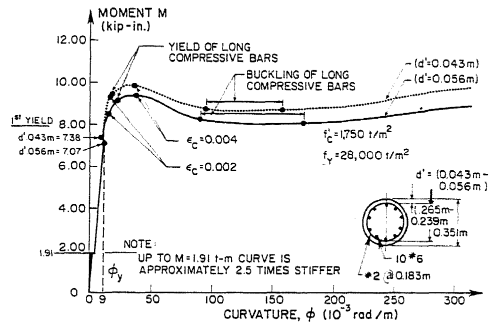


FIG. 3 EFFECT OF COVER THICKNESS ON M-Φ RELATION OF COLUMN CROSS SECTION UNDER $P = 12t$

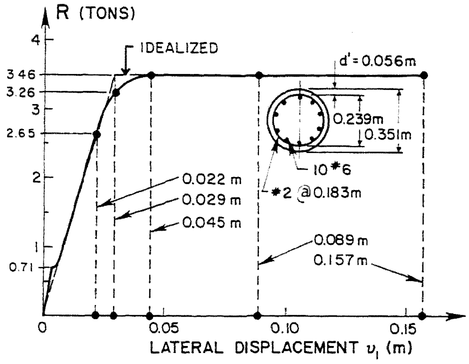


FIG. 4 LATERAL RESISTANCE FUNCTION, R vs. V_1

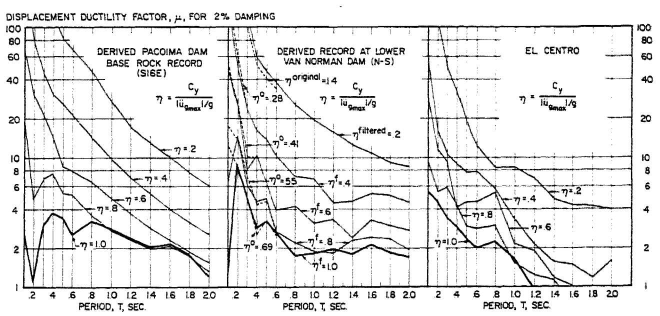
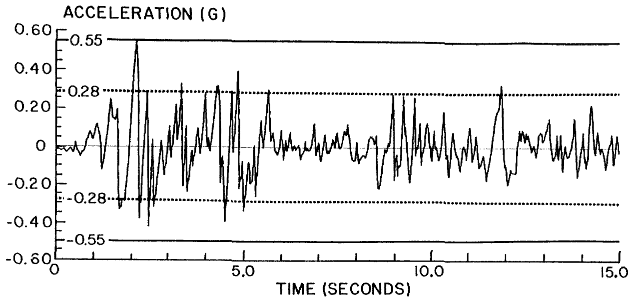
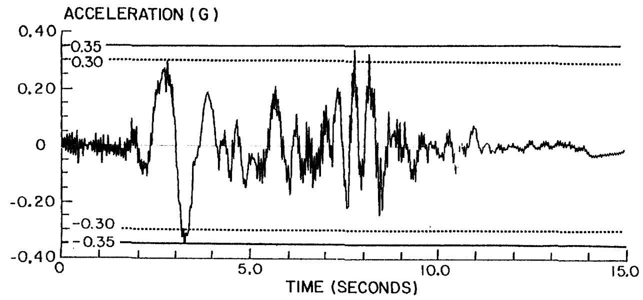


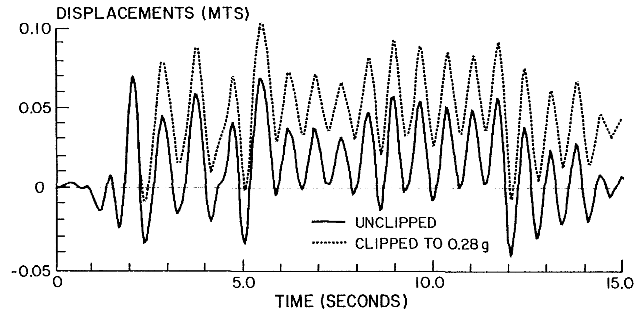
FIG. 5 INELASTIC RESPONSE SPECTRA FOR 2% DAMPING



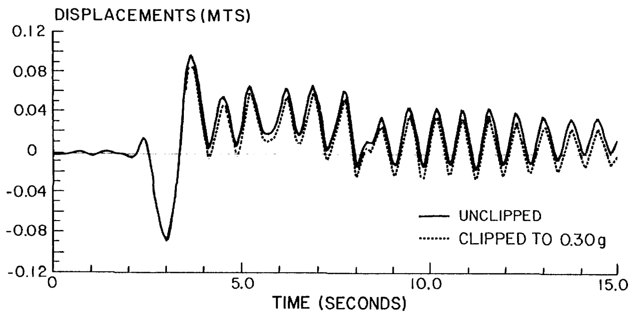
(a) EL CENTRO 1940 NORMALIZED TO 0.55g



(c) DERIVED PACOIMA DAM NORMALIZED TO 0.35g



(b) DISPLACEMENT RESPONSE OF CANOPY TO EL CENTRO NORMALIZED TO 0.55g: UNCLIPPED & CLIPPED TO 0.28g



(d) DISPLACEMENT RESPONSE OF CANOPY TO DERIVED PACOIMA DAM NORMALIZED TO 0.35g: UNCLIPPED & CLIPPED TO 0.30g

FIG. 6 EFFECT OF CLIPPING EL CENTRO NORMALIZED TO 0.55g & DERIVED PACOIMA DAM NORMALIZED TO 0.35g ON DISPLACEMENT RESPONSE OF CANOPY