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Prefabricated Portal Frames Subjected to Variable Repeatitive Loads

Cadres à portique préfabriqués soumis à des charges répétées variables

Vorfabrizierte Portalrahmen unter veränderlicher wiederholter Belastung

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Introduction

Adequate safety of a structure is the primary aim of a structural designer. Safety provisions after certain level of probability of failure of a structure have little significance in terms of real safety. Minimum load factors that ensure safety of a structure will yield maximum economy in structural design. Load factors which are successfully used as safe factors vary with specifications and some typical specifications are given in table 1.

Most loads on structures are variable repeated type of loads. Load factors which are dependable for static test should also be checked to ensure safety under repeated loads. Some investigations [7–14] on repeated loads on reinforced concrete structures indicate: a) Neither rotation nor the load carrying capacity of an R.C.C. beam is affected by several cycles of near ultimate loading. b) Repeated load causes progressive deformation, however, the deformations are moderated and increase at decreasing rate. c) Stiffness of R.C.C. beams decrease to a limited extent due to repeated loads but stabilize after several repeatitions of the load. It has been reported [15 to 17] that repeated loads decrease the bond capacity of the prestressed concrete beams. Research data on rotation capacity of joints in prefabricated construction under loads appears to be lacking [18].

Experimental Programme

Portal frame specimens: Each of the four portal frames selected for investigation was prefabricated with one beam and two column elements. The precast

Table 1. Typical Load Factors

No. Specification	Material reduction coefficient	Combined Dead and Live Loads		Load Factors Combined Dead, Live and Wind or Earthquake Loads		
		N_d	N_1	N_d	N_1	N_w or N_e
1 CEB-FIP [1][1])	0.9[2])	1.2	1.4	1.2	1.26	1.26
2 COMECON-SNIF [2] (USSR, East European)	0.9	1.1 to 1.2	1.4	1.1 to 1.2	1.26	1.26
3 MSZ [2] (Hungarian)	0.8 to 0.9	1.1 to 1.2	1.3 to 1.4	1.2	1.4	0.6
4 ACI [3] (USA)	0.9	1.4	1.7	1.4	1.5	1.5
5 Japan [4]		1.2	2.1 or	1.2 1.0	1.4 1.0	1.4 1.5
6 CP-[5] (UK)	1	1.4	1.6 or	1.25 1.40	1.25 —	1.2 1.4
7 IS [6] (India)	1	1.5	2.2 or	1.5 1.5	2.2 0.5	0.5 2.2

Where N_d, N_1, N_w and N_e are load factors for Dead, Live, Wind and Earthquake respectively.
[1]) The numerals in the brackets indicate the reference numbers.
[2]) Material reduction coefficient depends on probability of strength requirement.

column elements were of reinforced concrete while the beam element was of concrete which was post-tensioned after assembling. The beam element was provided with shear reinforcement in the shear zone such that the shear strength of the beam was atleast 1.2 times its bending capacity. The joints of each of the portal frame specimens had different strength capacities while the capacities of the beam and column elements were same for all the four specimens. 15 cm cube compressive strength of the concrete was 450 kg/cm^2. The yield strength of the mild steel was 2800 kg/cm^2 while the ultimate strength of the high tensile steel was 16 500 kg/cm^2.

The reinforcements required to develop the design strength of the joints (beam and column intersections) were placed in the beam and column sections in position. The beam and column elements were precast separately and were put together to form a portal. The joint reinforcement coming from the beam was welded to that of the column, and the concrete was placed at the joints and cured for 28 days. Fig. 1 illustrates the details of the reinforcement.

Loading: The specimens were laid flat on the structural floor over frictionless balls, and subjected to two point loading through test cylinders. Hinged column conditions were achieved through V block arrangement. The loading arrangements and other details are shown in Figs. 2 and 3. The test cylinders were monitored from a pulsator which can generate variable pulsating loads. Each specimen was first subjected to two cycles of static loading. Load

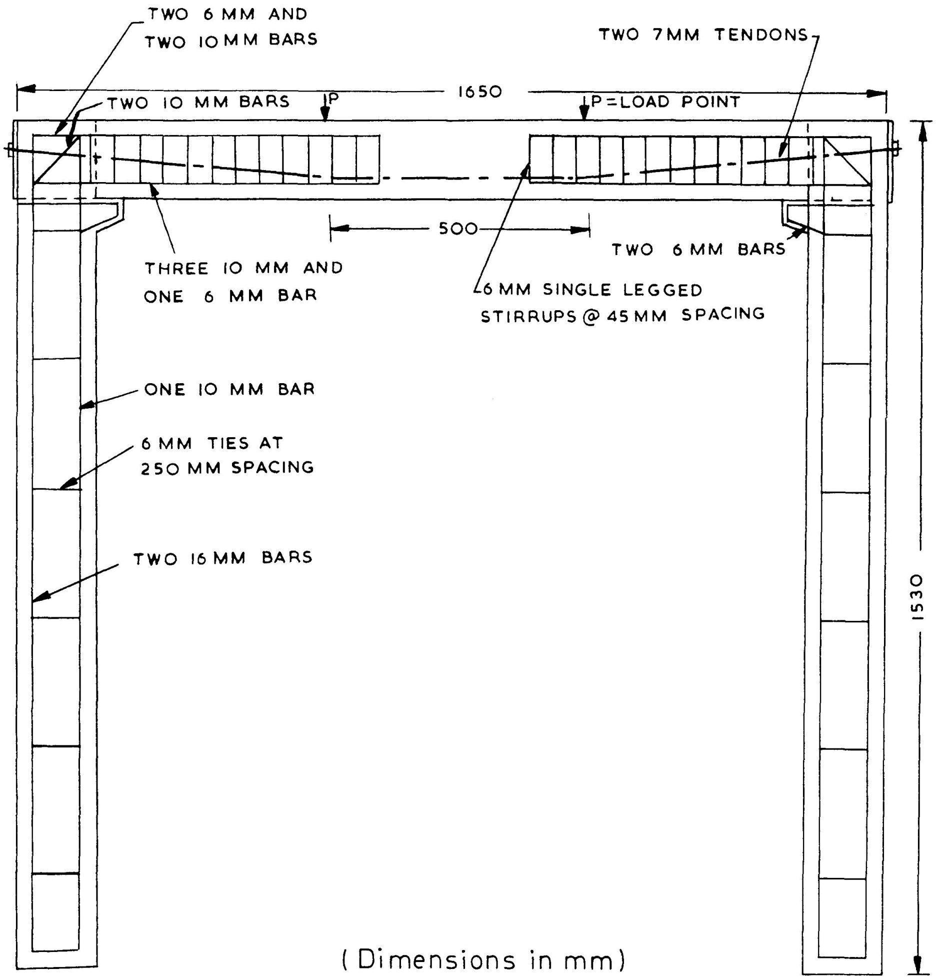


Fig. 1. Reinforcement details.

deflection measurements were made during these two cycles. The specimens were then subjected to variable pulsating loads of about 1.2 millions pulses. The specimens were then subjected two cycle, static tests and finally to failure.

Variable Pulsating Load: The variable pulsating load consists of three bounds: 1. the lowest bound corresponding to a certain assumed permanent load, 2. one of the two upper limits of the pulsating loads corresponds to an assumed normal live load and 3. the second upper limiting load which was applied less frequently than the first normal live load, corresponds to an extraordinary live load which is likely to occur for a limited number of times.

The design load criterion for the specimens was specified as

$$U = N_d D + N_{11} L 1$$

or

$$U = N_d D + N_{12} L 2 \qquad (1)$$

where

U = ultimate load (plastic collapse load of the frame),
D = dead load,

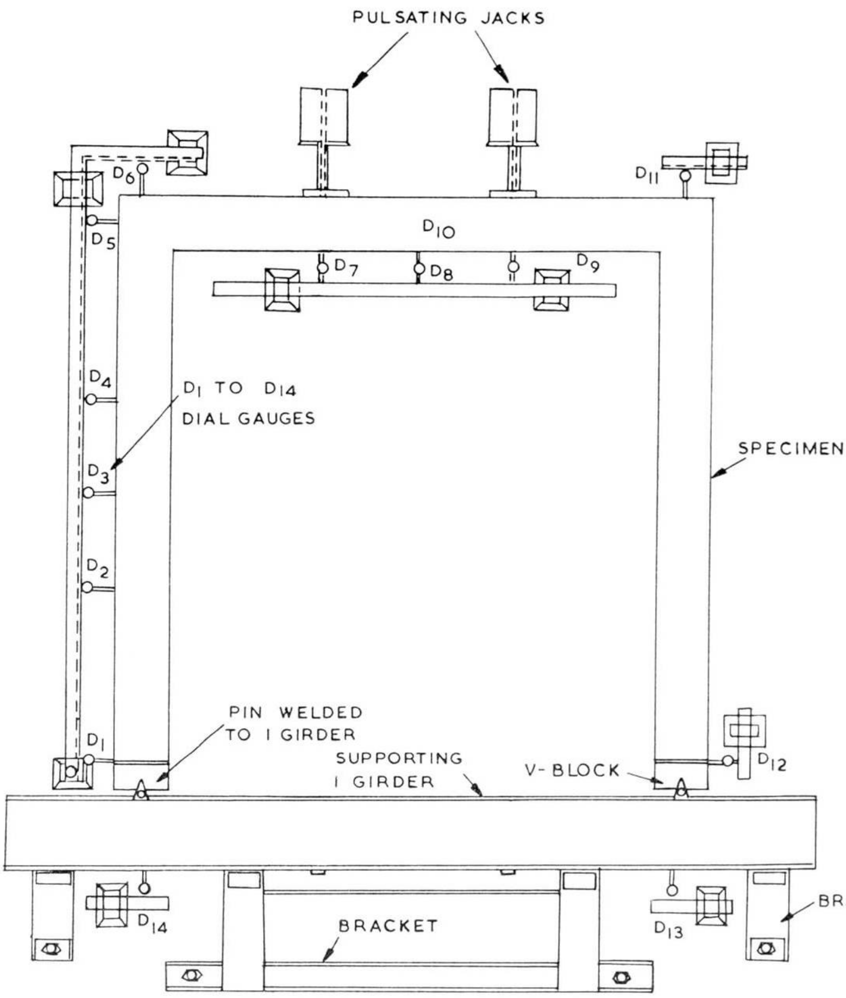


Fig. 2. Test setup.

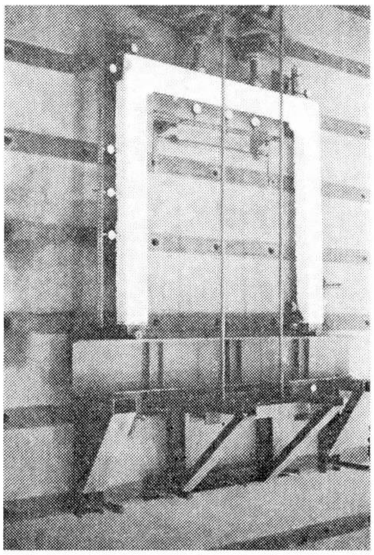


Fig. 3.

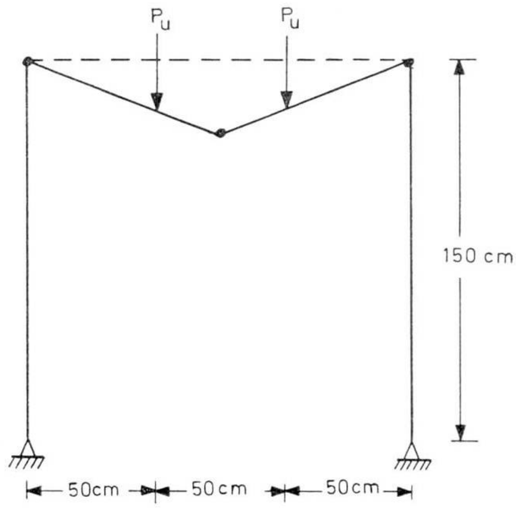


Fig. 4. Collapse mechanism.

$L1$ = normal live load,
$L2$ = live load which is likely to occur less frequently as compared to $L1$ ($L2 > L1$),
N_d = load factor for dead load,
N_{11} = load factor for live load $L1$,
N_{12} = load factor for live load $L2$, ($N_{12} < N_{11}$).

There are several possible combinations of dead and live load situations in actual structures. The present investigation selected four sets of relative values of dead and live loads. Reinforcement at the joints varied marginally while the prestressing force is kept the same in all the four specimens. The moment capacities of the beam and the joints are given in table 2. The moment capacity

Table 2. Moment Capacities and Collapse Load of the Specimens

Specimen	M_b (kg cm)	M_j (kg cm)	$P_u^t = (M_b + 0.9\,M_j)/50$ (kg)
1	101000	148000	4684
2	101000	148000	4684
3	101000	158000	4834
4	101000	124000	4252

of the beam was computed without applying any material reduction coefficient since the elements were precast. The moment capacity of the joint was reduced by 10 percent to account for the cast-in-situ joint while calculating the collapse load of the frame. The collapse load of the specimens which corresponds to a beam collapse mechanism is given by

$$P_u^t = \frac{M_b + 0.9\,M_j}{50}, \qquad (2)$$

where M_b = ultimate moment capacity of the beam at mid section,
M_j = ultimate moment capacity of the beam at the joint,
P_u^t = theoretical ultimate (collapse) load at one 3rd point.

Load cycle: The pulsating loads on the specimens were varied between D and $D+L1$ at the rate of 600 pulses per minute for 177 minutes. The upper limit of the pulsating load was then gradually increased from $D+L1$ to $D+L2$ in one minute. This peak load was maintained for one minute as upper limit while the lower limit was maintained at D. The upper limit load is again decreased gradually to $D+L1$ from $D+L2$ in one minute. This operation was once more repeated thus bringing total loading time to 360 minutes. The load was then completely reduced to zero and the specimen was left free from all the external loads for the remaining 18 hours of the day. This 24 hours period was counted as one full cycle of loading. The frequency of loads on actual structures will be less than 600 pulses per minute whereas the experiment had to be conducted at such a frequency so as to minimize the time required. The accelerated frequency of loading on the specimens may produce some fatigue effects. Such fatigue effects are minimized by giving a relaxation in the loading. Fig. 5 illustrates the variable pulsating loading programme used in the investigation. This type of pulsating loading was continued for more than one million pulses of the loads.

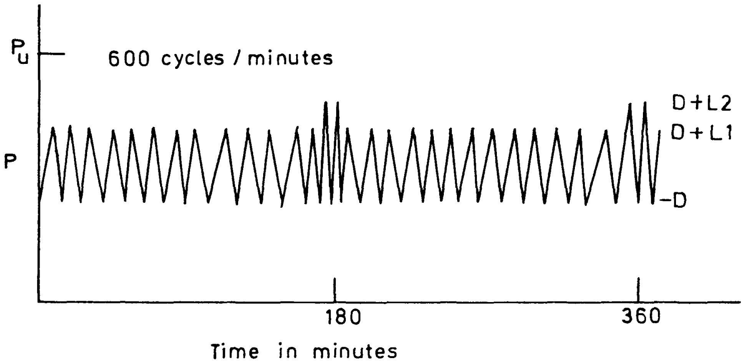


Fig. 5. Typical load cycle.

Each specimen was again subjected to two cycles of static loading after it had undergone the pulsating loading programme. Finally the specimen was subjected to a monotonically increasing load till failure. The total experimental programme can be summerized as:

1. Two cycles of static loading as a pre-pulsating load programme.
2. Pulsating load programme – loads varying between a fixed lower load and two upper load limits.
3. Two cycles of static loading as a post pulsating loading programme, and
4. Testing for total collapse.

Table 3 illustrates the theoretical ultimate loads and the load factors corresponding to the two live loads. Load factor for dead load was selected as 1.2 for all the specimens. The peak live load $L\,2$ was assumed to occur at 1 in 90 of the normal live load $L\,1$.

Table 3. Ultimate Load and Load Factors Without Applying Material Reduction Coefficient

Specimen (i)	P^t_u (kg)	N_{11}	N_{12}	$\frac{N_1(i)}{N_1(1)}$	P^e_u (kg)	$\left[\frac{Pe}{Pt}\right]_u$
1	4684	3.0	2.0	1.00	4800	1.03
2	4684	2.7	1.8	0.90	5150	1.10
3	4834	2.3	1.5	0.75	4730	0.98
4	4252	2.0	1.3	0.65	4800	1.13

P^e_u = experimental ultimate load.
(i) = number in the brackets in column 5 indicates the specimen number.

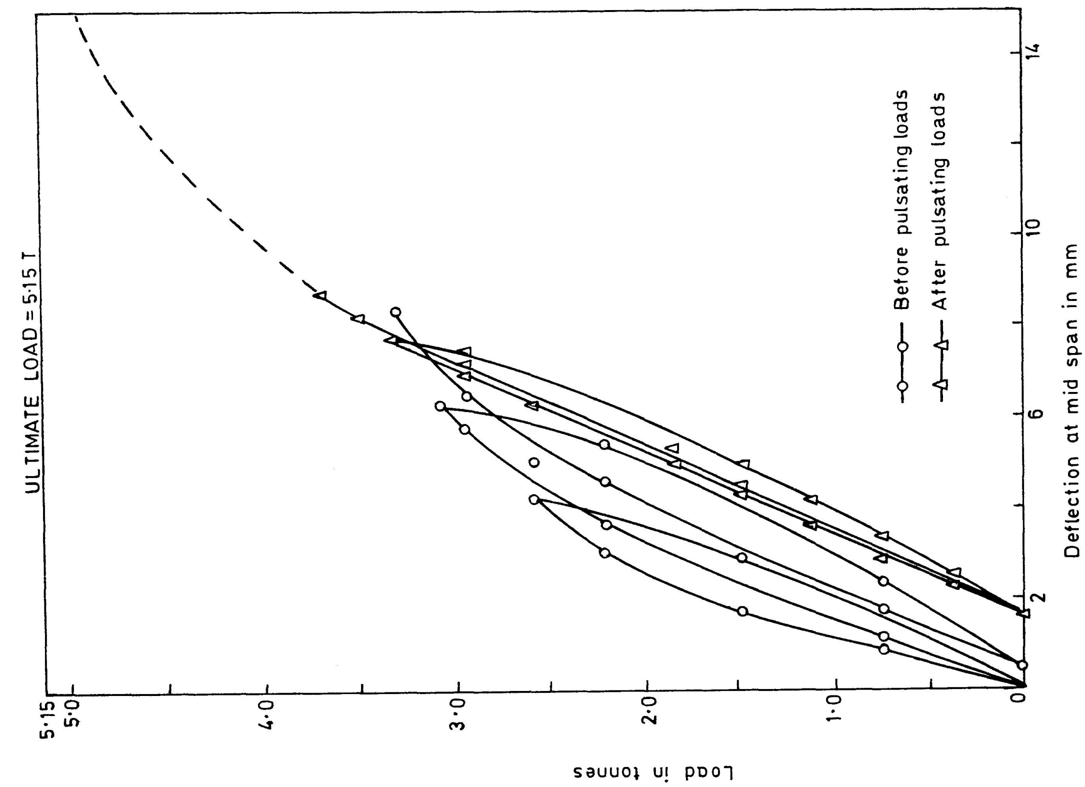


Fig. 7. Load-deflection curve of specimen 2.

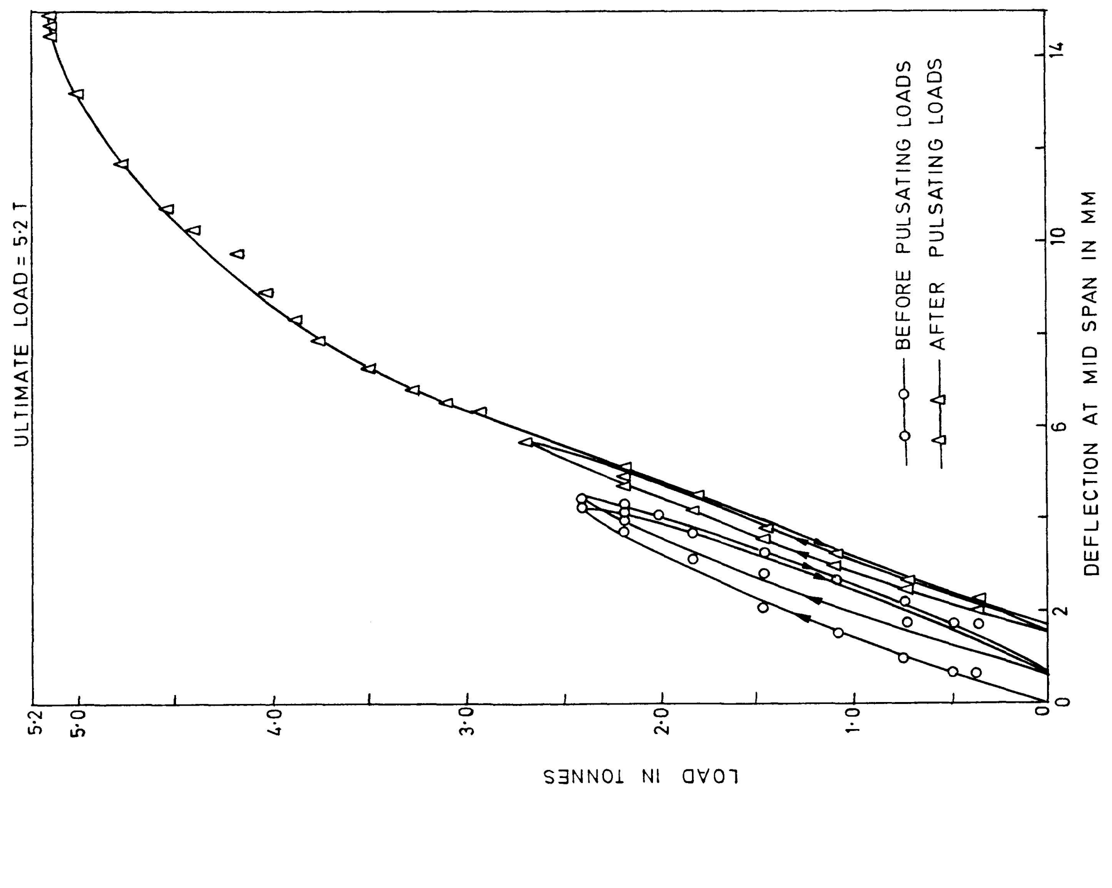


Fig. 6. Load deflection curve of specimen 1.

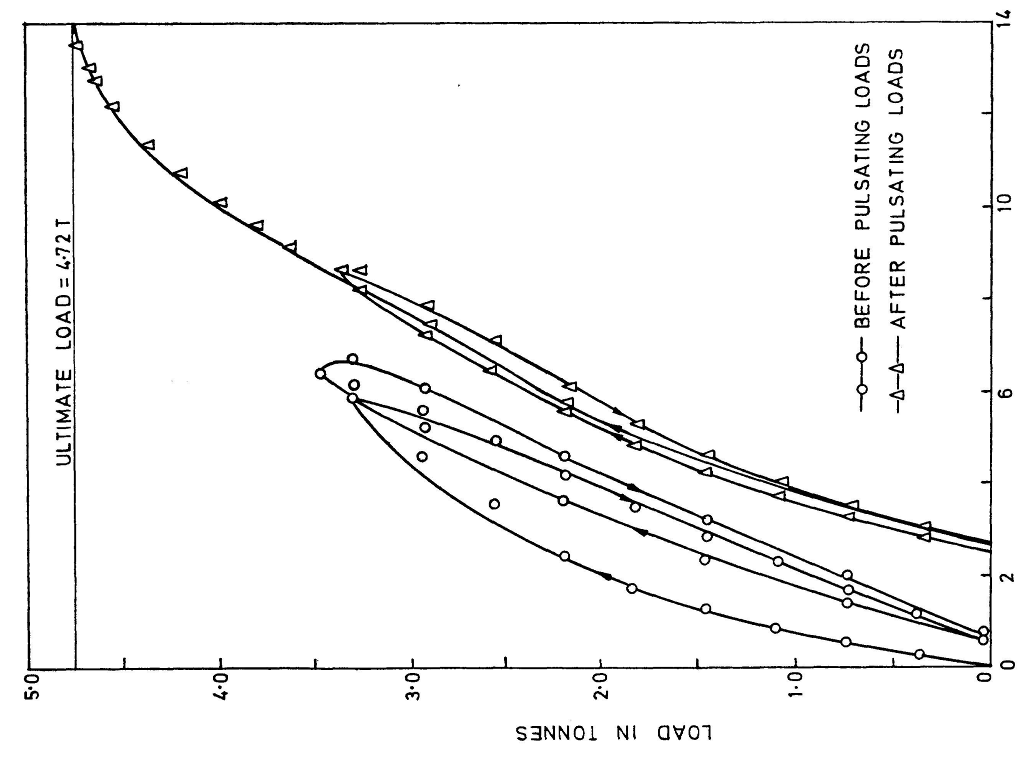


Fig. 8. Load deflection of specimen 3.

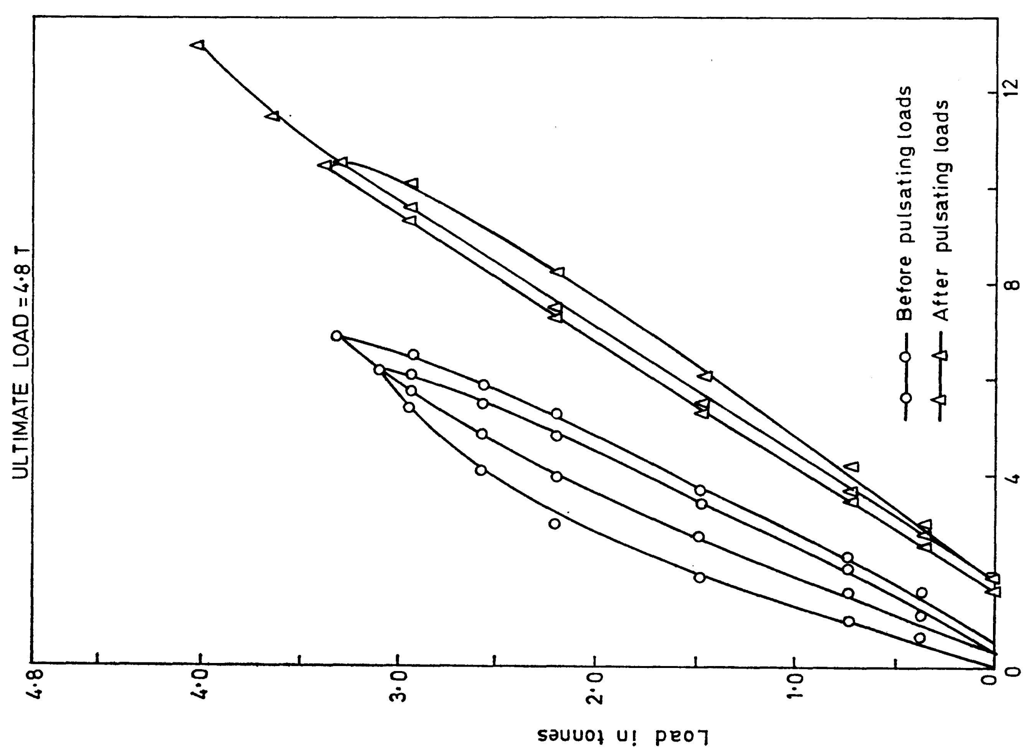


Fig. 9. Load-deflection curve of specimen 4.

Test Results

Deflection behaviour: Two cycles of static load deflection measurement of the specimens taken before and after the pulsating load are shown in Figs. 6 to 9. It can be observed that the area within the loop enclosed by the loading and unloading paths of any given cycle before pulsating load is about three times the corresponding area of the loop developed after the pulsating load. Therefore, the energy dissipation in cyclic loading after several load cycles decreased by at least seventy percent. Deflections of the frames at the peak loads before and after the pulsating loads are shown in Figs. 10 to 12. Similarly, the residual deflections at no load condition immediately before and after the pulsating load are also shown in Figs. 10 to 12. The dotted line in the figures

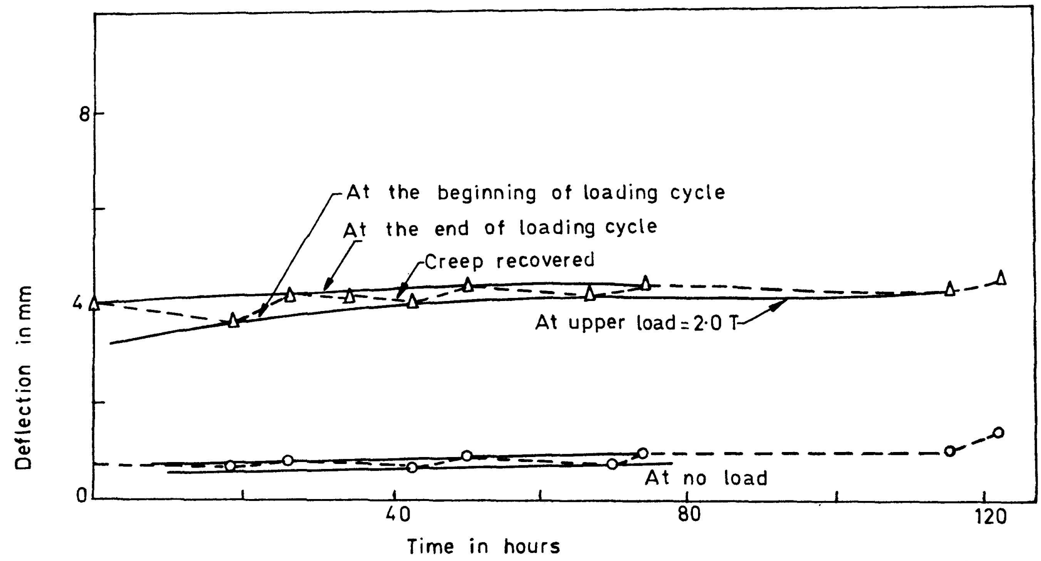


Fig. 10. Cumulative deflections of specimen 1.

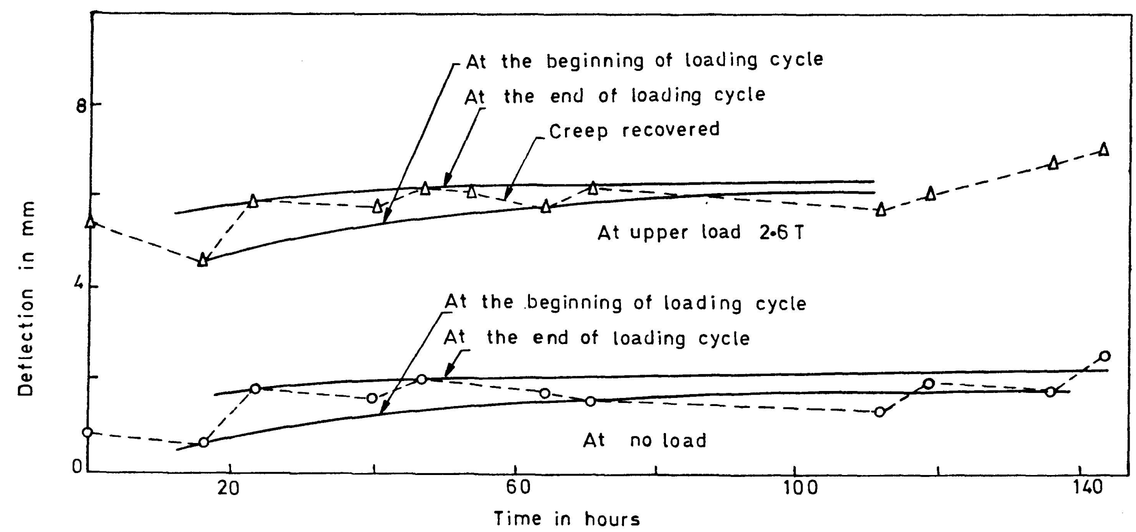


Fig. 11. Cumulative deflection of specimen 3.

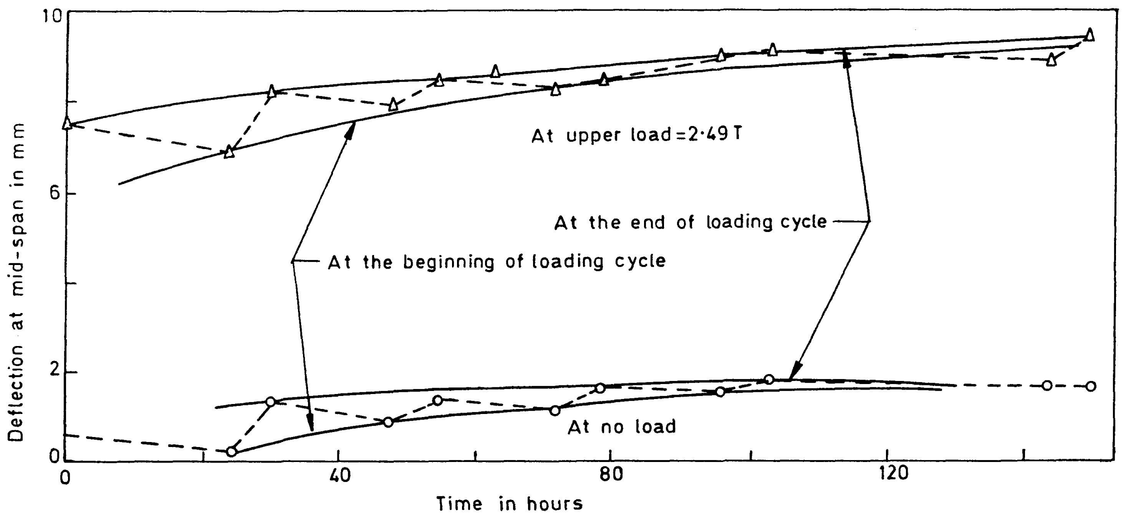


Fig. 12. Cumulative deflection of specimen 4.

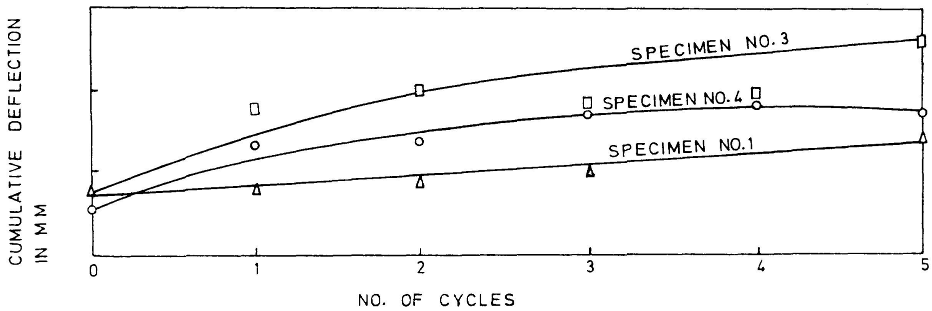


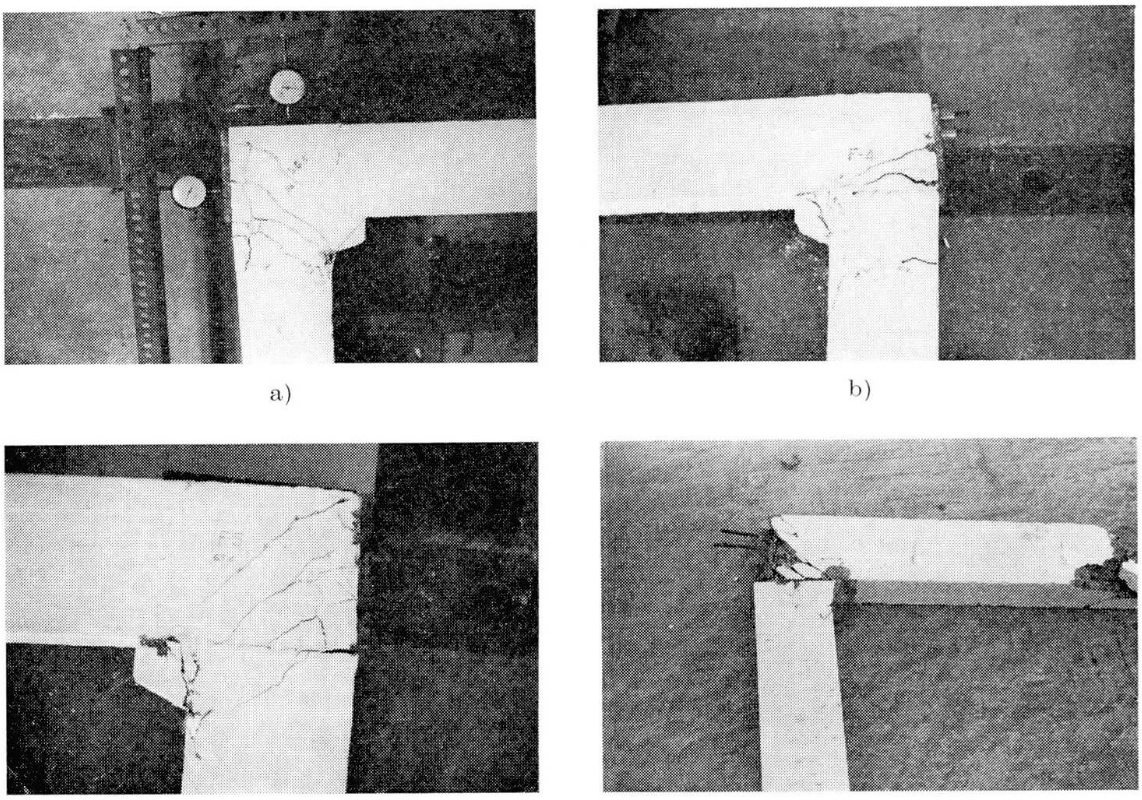
Fig. 13. Cumulative deflection V_s number of cycles.

indicates the sequence of the deflection measurements. Fig. 13 illustrates the cummulative increase in the deflection of the portal frames due to pulsating load. Following observations are derived from these figures.

1. Instantaneous increase in the deflection due to pulsating load is about 20 percent in the first cycle of pulsating load. However, this increase in deflection decreases about 10 percent in the subsequent cycles of loading.
2. Most of the increase in the deflection due to pulsating load was recovered during the relaxation period.
3. There is a cumulative increase in the deflections of the frames. The total cumulative residual deflection of the specimen 1 was about 25 percent of the deflection at the peak load where the live load factor at peak load was 2. However, the cumulative residual deflection of the specimens 3 and 4 were 30 to 40 percent of the deflection at the peak load where the load factors at peak loads were 1.5 and 1.3.
4. The deflection behaviour of the frames even after the application of the pulsating load was very similar to that of the beam before pulsating load even though wide cracks appeared in the frames during peak loads.

Cracking

Cracking pattern in all the four samples was same. The first cracks which occured at the mid span of the beam, were at 40 to 50 percent of the ultimate load while cracks in the joint occured at 55 to 65 percent of the ultimate load. The cracks in the beam were closed completely while those at the joints closed partially. The failure of the joint had occured on the column face even though the designed capacity of the joint on the column face was slightly higher than that on the beam face. Cracking strain level of the joint on the beam face was much higher than that on the column face with the result that the cracking was initiated and propagated on the column face which ultimately lead to failure. Diagonal cracking in the joints had appeared before failure but the actual failure was by crushing of the concrete on the column face. Fig. 14 illustrates the cracks in the joint which widened during the static test.



a) b) c) d)

Fig. 14.

Ultimate Load Capacity

The ultimate loads of the specimens after several (more than one million) application of the variable pulsating load were very close to the theoretical loads based on without any material reduction coefficient. Since the specimens were cast and cured in the laboratory conditions, it is probably not necessary

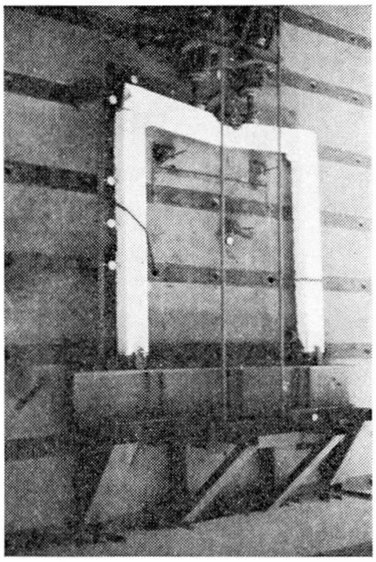


Fig. 15.

to apply any reduction coefficients. All experiments either static or pulsating conducted in the structural engineering laboratory [19] checked closely with the theoretical results without any reduction factors. The specimens exhibited good ductility even after the application of the variable pulsating loads. There was complete redistribution of moments and the specimens failed in simple beam mechanism. The reinforcement provided at top and bottom faces of the joint might have been responsible for ductility and redistribution of the moments.

Conclusions

The following conclusions are arrived at in the present investigation on prefabricated portal frames. All the specimens were subjected to about 1.2 million repeatitive type of normal live loads and 1.33×10^4 repeatitive peak live loads. The load factor adopted for dead load was 1.2.

1. Cumulative residual deflections of the order of $\frac{1}{1000}$ to $\frac{1}{600}$ span were caused by the repeatitive loads. Worst strained specimens had load factor for the peak live load as 1.3. In other words, the specimens were subjected to 1.33×10^4 repeatitions of 75 percent of its ultimate load capacity without damage.
2. The residual deflections immediately after the peak load of the first few cycles were quite considerable. However, 70 to 80 percent of the instantaneous residual deflections were recovered within few hours of unloading.
3. The load deflection pattern of the specimens was not much affected by the repeatitive loads except the loading and the unloading paths have come closer to each other after the repeatitive load.
4. The cracks first appeared in the beam element at 40 to 50 percent of the ultimate capacity while the joints showed cracks at 55 to 65 percent capa-

city of the frame. Most of the cracks disappeared after the removal of the loads. The cracks at peak loads were seen very clearly as the number of the repeatitive cycles increased.

5. The specimens exhibited good ductility even after the pulsating load and a complete redistribution of the moments was indicated in the plastic collapse mechanism.
6. The ultimate load capacity of the portals was not affected by the pulsating loads. The experimental results checked very closely with the theoretical calculations.
7. A reduction of atleast 10 percent in the moment capacity of the cast-in-situ joints be accounted in the collapse load calculations. No material or manufacturing reduction coefficient appears to be necessary for precast elements.
8. If cracks have not to appear at all in the life of the structure, load factor for any live load should not be less than 1.8. If observable cracks are permitted during peak loads, then load factor of 1.5 for the peak load is acceptable. A load factor of 1.3 for peak load is acceptable if the permissible cracks widths be of order of 0.4 mm.

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Summary

The paper presents behaviour of prefabricated portal frames subjected to variable pulsating loads. The deflection and ultimate strength of the frames was not affected by the pulsating loads. A load factor of 1.5 for peak load did not cause much cracking while a load factor of 1.3 for peak load produced wide cracks in the posttensioned beam element and also at the joints on the column face.

Résumé

Cette contribution traite du comportement de cadres à portique préfabriqués soumis à des charges variables. La déflexion et la résistance ultime des cadres n'étaient pas affectées par les charges variables. Un facteur de charge de 1,5 pour charge maximum a causé peu de fissures tandisqu'un facteur de charge de 1,3 pour la charge de pointe a provoqué de nombreuses fissures dans l'élément préfabriqué de la poutre et également à ses jointures avec les colonnes.

Zusammenfassung

Die Arbeit behandelt das Verhalten vorfabrizierter Portalrahmen unter veränderlicher wechselnder Belastung. Die Durchbiegung und zulässige Beanspruchung wurden durch die wechselnde Last nicht beeinflusst. Ein Lastfaktor von 1,5 für Spitzenlast verursachte nicht viele Risse, wogegen ein Lastfaktor von 1,3 für Spitzenlast ausgebreitete Rissbildung sowohl im vorfabrizierten Balkenelement als auch an dessen Verbindungsstellen mit den Stützen zur Folge hatte.