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Structural Performance Evaluation of Innovative Building Systems

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Felix Y. Yokel and Norman F. Somes

Considerable attention has recently been focused on the development of performance criteria. Performance Criteria are presently used in the U.S. by the Building Research Division of the Institute for Applied Technology, National Bureau of Standards, to evaluate innovative building systems.

While building codes and design standards are usually related to specific building materials and design solutions, performance criteria are derived from user requirements and are independent of specific technological solutions.

Many attributes inherent in traditional building systems which are acceptable to the user may not be present in untried innovative systems. The Performance Criteria therefore address themselves to many aspects of structural performance which are not considered in present codes and design standards.

The successful application of performance criteria depends on the feasibility of evaluating compliance.

Performance can be evaluated by analysis by judgement based on past experience, or by physical simulation. Deterioration with time must be considered, and performance criteria are viewed as minimum requirements which should be met at any time during the service life of a structure. An example is presented where physical testing was used to evaluate the performance of a high rise housing system.

Key words: Building; connections; housing; neoprene; performance criteria; performance evaluation; performance testing; reinforced concrete; standard tests; structure; testing.

1. Performance Criteria Versus Design Standards

Considerable attention has recently been focused on the development of performance criteria [1, 2, 3].[1] Performance criteria, which are presently used in the United States by the Building Research Division of the Institute for Applied Technology, National Bureau of Standards, have been discussed in a paper presented at the 5th CIB Congress in June 1971 [4].

Performance criteria define the required attributes of a building in terms of the functions the building has to perform. More specifically, attributes are defined in terms of the user requirements of safety, activity support, acceptable maintenance cost, absence of stress and anxiety (comfort) and visual acceptability.

The structural attributes thus defined can be identified within broad categories of limits for strength, stiffness and rigidity, and resistance to local damage. These limits can be related to the required performance which is entirely independent of any particular design concept and material application.

Because of this independence from specific hardware solutions, the criteria cover many aspects of structural performance not considered in present building codes and standards. These additional provisions are necessary, since many attributes which are inherent in traditional systems are not necessarily present in untried innovative systems. Included are requirements for load capacity which take strength variability into account, provisions for strength under cyclic loading and prevention of progressive collapse, criteria limiting vibrations and criteria for resistance to local damage under various concentrated and impact loads. Also, now under consideration, are criteria for structural ductility (the ability to undergo large irrecoverable deformation without fracture) to be applied in particular in seismically active regions.

*Presented at the 1st CIB Symposium on Tall Buildings (S-41), Moscow, U.S.S.R., October 1971.

[1] Figures in brackets designate literature references.

2. Methods of Performance Evaluation

The successful application of performance criteria depends on the feasibility of determining compliance with the criteria. This determination is made by a performance evaluation which is carried out using one or a combination of the following methods:

(1) analysis;
(2) professional judgement;
(3) physical simulation.

Traditionally, structural adequacy is determined by analysis which includes the use of previous design experience. However, when innovative untried designs are evaluated analysis by calculation alone is not always possible. In particular, it is difficult to predict certain aspects of structural behavior, such as long-term strength and deflections, ductility, damping of vibrations, and behavior under cyclic loading.

In such cases, it may sometimes be possible to use professional judgment and draw conclusions from past in-service experience or previous test results.

If neither analysis nor professional judgment can provide satisfactory answers, physical simulation must be used to supplement the analysis.

3. Guidelines for Physical Simulation

Physical simulation is a test procedure designed to simulate the response of an actual structure to various loading conditions. Since it is required that the structure satisfy the performance criteria at any time during its service life, physical simulation must be interpreted to account for the effects of structural deterioration with time. This implies that in many cases the performance of a structure during the early years of its life must exceed the performance level required by the criteria. The margin, by which the required performance must be exceeded, depends in each instance not only on the materials and the systems used, but also on the uncertainty in judging the aging effects on performance. This uncertainty tends to be greater for systems with little or no service record.

The criteria that were set for physical simulation require:

(1) selection of critical subassemblies for testing;
(2) use of critical loading conditions;
(3) allowances for, or simulation of, the effects of the service-life environment;
(4) consideration of variability in performance.

These four points need further discussion;

(1) Selection of critical subassemblies for testing.

If structural response is simulated by full-scale testing, it is usually not possible to test more than a subassembly representing a small part of the structure. Such subassemblies should be critical in their configuration (e.g., using the largest floor spans or the least number of shear walls that would be used in an actual structure). Where foundation and joint fixity tend to increase load capacity and stiffness, it is important to consider potential loss or absence of foundation fixity by settlement or adverse soil conditions and a possible decrease in joint fixity by system slack and loss of tightness, deficiencies in fabrication, repeated loading, material deterioration, or loss of composite action. Choice of critical subassembly is particularly important when prefabricated building systems are evaluated, where variations in architectural layout may result in reductions in, or unsymmetrical arrangement of, shear walls available to resist lateral loads or other critical load-supporting elements. In such cases the introduction of openings for doors, windows, ducts, or staircases in critical locations may also result in weakening of shear walls and reduction of vertical-load and diaphragm (in-plane loading) capacity of floors.

Consideration must also be given to the interaction of loadbearing and non-loadbearing elements, such as the interaction between structural frames and non-loadbearing partitions. In some instances, non-loadbearing elements contribute to the strength and stiffness of the loadbearing structure. However, the lack of compatibility in stiffness may cause severe damage to the non-loadbearing elements. If unaccounted for, the stiffening effects of non-loadbearing elements may also reduce the energy-absorbing capability of the structure under seismic loads.

(2) Use of critical loading conditions.

Critical loading conditions do not occur only as a consequence of critical load combinations acting on the structure at critical locations. Other considerations are also important. An example is the added moment induced by eccentricities of compressive loads acting on vertical members. Such eccentricities may be aggravated by incidental construction misalignment as well as by permissible tolerances of fabrication and erection, by instantaneous and time-dependent deflections and by lateral drift caused by wind forces or foundation settlements.

(3) Allowance for, or simulation of, the effect of the service-life environment.

The simulation of the effects of service life deterioration presents one of the most difficult problems in performance evaluation. The effects are particularly difficult to assess when no service records are available.

The main difficulty arises from the necessity of simulating in a test of reasonably short duration environmental effects that occur over the service-life period. Examples of environmental conditions that cause deterioration are moisture and freezing, temperature changes, ultraviolet light, chemical reaction, long-term loading, and cycles of repeated loading.

These conditions may cause deterioration of materials, loss of strength in structural composites (particularly when adhesives are used), loss of stiffness in structural joints, weakening of members by fatigue, and load transfer from one structural element to another by differential creep, (i.e., transfer from concrete to reinforcing steel) and by effects of temperature and moisture.

All the above effects are reasonably well understood in conventional construction and can be predicted by analysis. However, in innovative systems and material applications, frequently only performance testing can provide a basis for prediction.

(4) Consideration of variability in performance.

Variability must be considered, particularly in the evaluation of strength (load capacity).

Recently, considerable research has been devoted to the introduction of probabilistic concepts into structural design [5]. At the present time, the professional concensus in most countries is that the state of the art has to be further advanced before such a step can be taken.

However, when the structure considered is very different from conventional construction, particularly with respect to the application of structural materials, presently-used margins of safety which are associated with particular construction materials (concrete, steel, masonry, etc.) may not be applicable. In these cases, due consideration of strength variability provides the only rational basis for setting safety margins.

The criteria require:

(1) that the structure not fail under certain required "ultimate" loads and load combinations, which are service loads[2] multiplied by load factors. The factors insure that these "ultimate loads" have a very low probability of occurrence. The requirement is satisfied if a suitably large percentage of a population of structures would not fail under the required ultimate loads.

(2) that the least credible strength be sufficient to preclude failure under any combination of service loads.

Those requirements, in general, are deemed to be satisfied by structures which are designed by present widely accepted design standards and which meet the requirements of the safety margins contained in these standards. However, in order to determine whether innovative structures, which cannot be designed by present standards, meet both requirements, some basis must be available to determine or estimate their strength variability.

Strength variability provides the basis for determining by how much the average or "design" ultimate load capacity should exceed the load capacity required by the criteria. A strength excess is required in order to insure that the probability be suitably high that any building actually constructed have at least the load capacity required by the criteria. When performance testing provides the basis for determining average or "design" load capacity, allowance must also be made for the possibility that the test sample selected had higher-than-average strength. Thus, the average load capacity of the test sample must exceed the design load capacity. The margin by which the design load capacity must be exceeded can be determined by sampling theory, and depends on the number of specimens contained in the test sample.

The above-discussed relationships are illustrated in a very simplified form in figure 1. In this figure it is assumed that the variation of strength in the population of structures built follows approximately a normal distribution. The coefficient of variation of single structures with respect to the mean strength of the population of structures is v, and the coefficient of variation of the mean of a test sample (which is larger than 1; say n) with respect to the mean of a population composed of

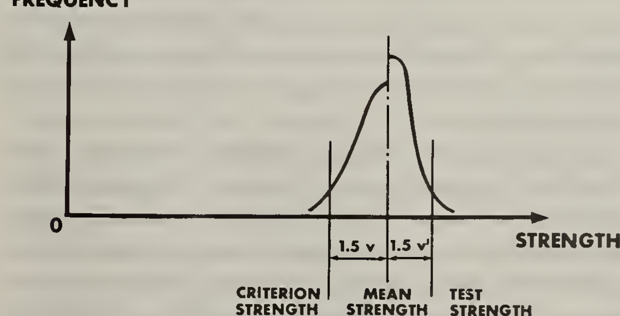

FIGURE 1. Relationship between strength variability and required load capacity.

[2] Service load is a maximum load likely to occur during the service life of the building

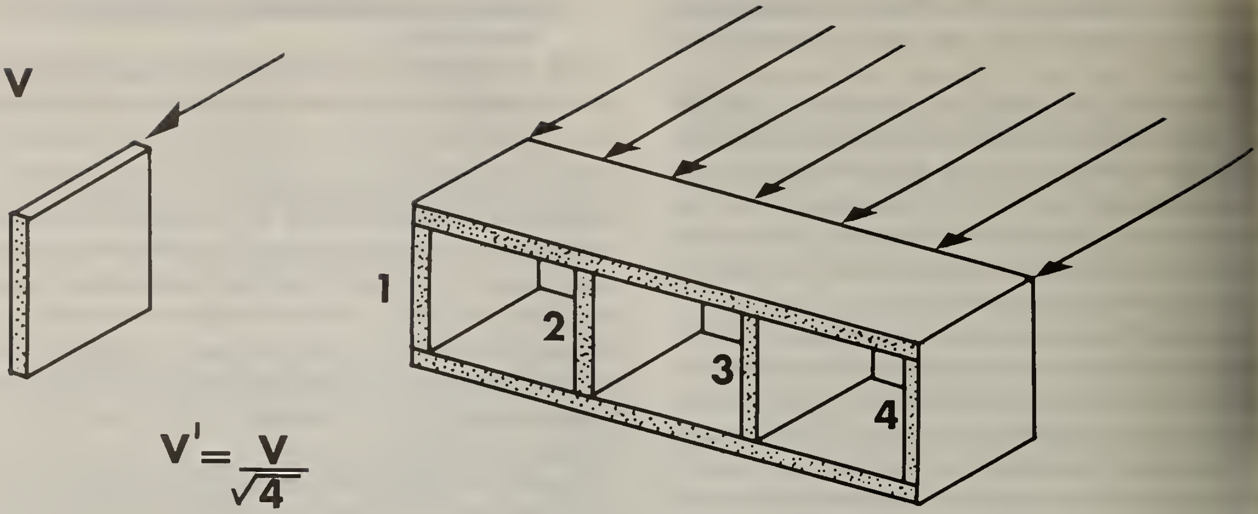

FIGURE 2. Effect of strength variability of components on the strength variability of the structure.

test samples consisting of n structures each is v'. The relationship shown in the figure corresponds to the requirement that approximately 95 percent of the population have at least the strength required by the criteria, and that there be about a 95 percent probability that the structures tested have at least the mean strength required to meet the criteria.

The effect of the strength variability of a component or subassembly on the strength variability of the complete structure is illustrated in figure 2 in a very simplified form. If the coefficient of variation of the strength of the single shear wall to the left in figure 2 is v, then the coefficient of variation of the assembly of four shear walls, v', illustrated to the right in the figure will be $v/\sqrt{4}$, provided the walls are ductile enough to develop and sensibly maintain their capacity before failure occurs and that the strength of the floor diaphragm corresponds to the failure mechanism. On the other hand, if load is transmitted vertically through many successive loadbearing members, and the failure of one member constitutes a failure of the structure, the probability of failure in the structure is greater than the probability of failure in a single member, subjected to similar loading.

Examples where strength variability has been considered include evaluation of untried structural adhesives, various structural composites including glass reinforced plastic or metal stress-skin structural components with paper honeycomb cores, and several instances of unusual jointing techniques or structural configurations in precast reinforced concrete structures.

4. Standardization of Performance Tests

It is essential that a performance evaluation be objective. Ideally, objectivity in the evaluation of compliance with any criterion requires a measurement method that cannot be influenced by the individual performing the measurement.

In practice, such objective methods are difficult to develop, since frequently many variables contribute to produce a certain attribute. All of these variables are not always measurable. A case in point is the previously discussed strength variability. Obviously, it would not be feasible to measure variability by testing a sufficient number of full-scale buildings to destruction. Indirect means must therefore be used which include not only tests, but also estimates of the magnitude of various parameters that cannot be reasonably determined by testing. In these cases, a set of guidelines takes the place of a standardized measurement method.

In other, more simple cases, efforts are now underway to develop standardized performance tests. Such tests have the advantage of being consistent and reliable and providing guidance to developers of innovative systems. The importance of standardizing tests is illustrated by the following example. Figure 3 shows the apparatus used for impact tests on floors. The leather bag was prepared in accordance with the standard procedure described in the American Society for Testing Materials (ASTM) designation E-72 [6], which specifies tests for floors and walls. However, since the criteria for impact load require a higher impact energy than that envisioned when the ASTM procedure was

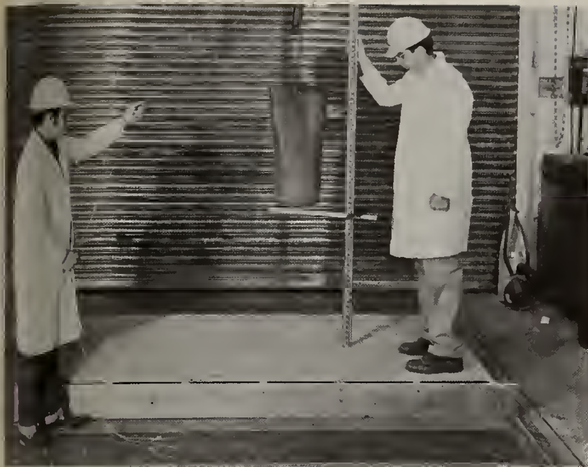

FIGURE 3. Apparatus for impact tests on floors.

developed, the bag was strengthened and filled with a mixture of sand and lead shot, rather than only sand as specified in ASTM designation E-72.

Various experiments that were performed indicated that the impact resistance developed by the floor systems not only depended on the impact-energy applied but also varied with the weight of the bag and the ratio at which sand and lead shot were mixed. The conclusion was that only a standardized test procedure would yield consistent and dependable results.

Another effort is underway to develop a standardized test for transient vibration. The criterion for transient vibrations of floors is presently expressed in terms of the rate of decay as a function of time, based on data which indicate the vibration induced by human activity, persisting at a perceptible level for less than 1/2 second, will not be disturbing to occupants [7]. However, further research has been initiated to consider also the variables of frequency and of displacement amplitude.

Figure 4 shows a test set up that was developed to measure floor vibrations. The equipment consists of a 25-lbm (11.34 kg) bag filled with sand and lead shot, a bag-release device mounted on a tripod for releasing the bag from a height of 3 ft (0.914 m) and a displacement transducer (linear variable differential transformer, LVDT). The LVDT is attached to a rigid beam of adjustable length which is positioned to span between two opposite walls.

The output of the LVDT is recorded by a recording oscillograph, presently equipped with a 600-Hz response galvanometer. Traces of the response are recorded on photosensitive paper fed through the oscillograph at a rate of 4 in (10 cm) per second. Figure 5 shows typical traces of displacement amplitudes obtained in a test.

FIGURE 4. Apparatus for measurement of transient floor vibrations.

Standardization of the above-mentioned tests is important, and efforts are also underway to standardize many other test procedures, among them concentrated-load tests on floors (fig. 6), impact tests on walls, environmental conditioning of test specimens, evaluation techniques for structural adhesives, and evaluation of structural ductility.

5. A Case History of Performance Evaluation

Figure 7 shows a housing system consisting of lightweight concrete modules, constructed in a checkerboard pattern. Several structures up to 17 stories in height have been erected, and a 22-story structure, using modules of different dimensions, is now in the planning stage. The modules for the 22-story structure are approximately 13 feet (4.0 m) wide, 52 feet (15.8 m) long and 8 feet 7 inches (2.6 m) high and weigh approximately 92 kip (41,700 kg).

The modules consist of a monolithically-cast ceiling-and-wall bent and separately-cast floor slab. The ceiling-and-wall bent derives its support from four ribs, each consisting of two columns with a horizontal beam connecting the columns at the ceiling level. Between these ribs there are three inch (76 mm)-thick infill reinforced concrete walls, and a four inch (100 mm)-thick infill reinforced concrete ceiling. The floor is also a four inch (100 mm)-thick reinforced concrete slab.

Figure 8 shows the assembly of the modules in more detail. One individual module is shown in figure 9.

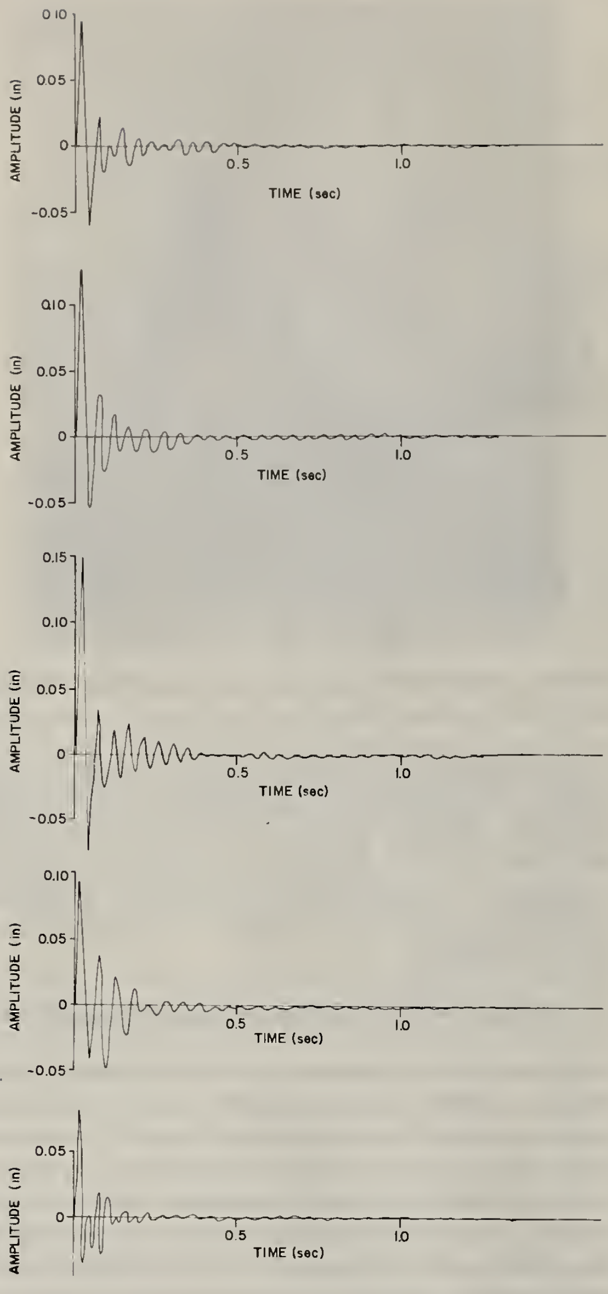

FIGURE 5. Displacement-amplitudes of transient floor vibrations.

Modules in successive stories are connected to each other by connecting the columns of the stiffening ribs in the modules. This is illustrated in figure 10. The checkerboard pattern of modules is closed at the end walls by inserting a special precast wall-column assembly which matches the sidewalls of the modules, and which is inserted at alternate story levels.

Figure 11 shows the connection details. In figure 11(a) the reinforcement detail at the column end is shown. In this case, it is necessary to transmit the full

FIGURE 6. Apparatus for concentrated load test on floors.

column load through a plain concrete bearing, with only one grouted dowel passing from one column to the next. The bearing stress on plain concrete permitted in U.S. design standards used at the time of the evaluation (ACI 318-63 [8]) was 0.25 f'_c, where f'_c is the 28-day concrete compressive strength. The concrete stress permitted for ultimate load in the proposed revision of ACI 318-63 [9] was $0.7 \times 0.85 f'_c$. In either case, in accordance with these standards, only a small fraction of the load that the column is capable of supporting could be transmitted through the connection. In order to increase the concrete compressive strength at the connection, additional reinforcement ties (loops), were proposed, as shown in figure 11(a). It was reasoned that these ties will increase the compressive strength of the concrete at the column face by providing confinement.

Figure 11(b) shows the detail at the connection. To accommodate structural tolerances, and also to minimize local stress concentration, a 1/4-inch (6.35 mm) thick neoprene pad was proposed to be inserted between two successive columns (cast in place or grouted connections were avoided for reasons of economy and ease of erection.)

Since Poisson's ratio for neoprene is higher and the modulus of elasticity is lower than that for concrete, it was anticipated that the neoprene pad would exert a radially tangential shear force on the concrete face causing tensile stresses and thereby counteracting the beneficial effect of the confining reinforcement ties. Quantitatively these effects could not be predicted by analysis, and past experience with concrete bearings on neoprene was confined to stress levels below 1000 psi (0.7 kgf/mm²). Structural analysis therefore had to be supplemented by testing.

FIGURE 7. Modular reinforced concrete housing system.

FIGURE 8. Detail of stacking of modules.

FIGURE 9. Individual module.

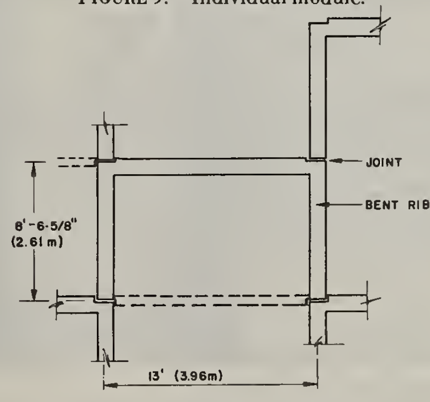

FIGURE 10. Stacking of module-ribs.

Figure 12 (left) shows one of the test specimens used. It consisted of two half-columns and a connection between them. Load was transferred to the half-columns through steel plates which were welded to the main reinforcement. The reinforcing arrangement of a half-column is shown in figure 12 (right). Most specimens were loaded axially to failure in a 600-kip (272,000 kgf) testing machine. Some specimens were loaded at a vertical-load eccentricity equal to 1/10 of the column thickness. The specimens were instrumented by a dial gage to measure the compression of the bearing pad, as shown in figure 12 (left). The figure also shows a displacement transducer mounted hori-

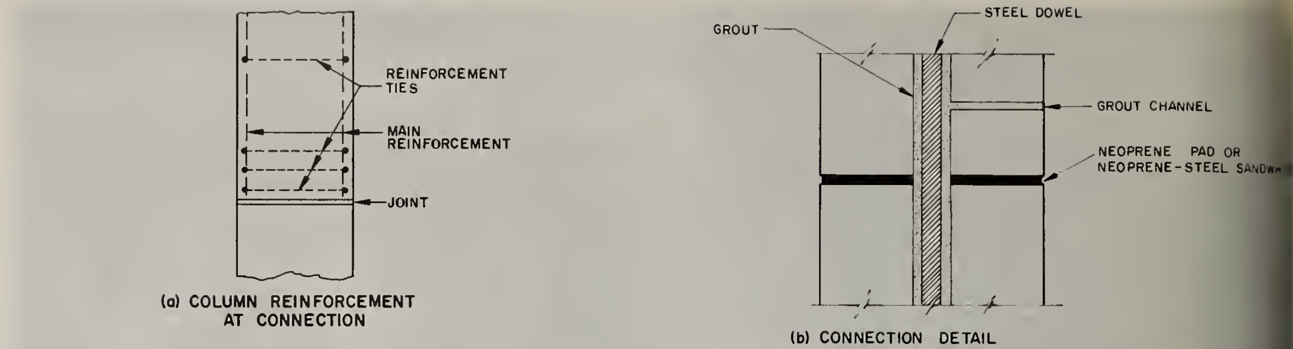

FIGURE 11. Detail of column connection.

zontally on the left column face. Two such transducers were mounted on mutually perpendicular column faces to identify the onset of concrete cracking by measuring the increase in the width of the column face.

The following interface conditions were tested:

(1) a 1/4-inch (6.35 mm) neoprene pad as proposed;
(2) a 3/4-inch (19 mm) thick bed of non-shrink high strength grout;
(3) a 1/4-inch (6.35 mm) neoprene pad, with low-friction material inserted between the neoprene and the column face (to limit the transmission of radially tangential shear forces);
(4) a 1/4-inch (6.35 mm) thick neoprene pad set between two 1/8-in (3.17 mm) thick stainless steel plates;
(5) a 1/4-inch (6.35 mm) thick neoprene pad set between two 1/16-inch (1.58 mm) thick stainless steel plates.

The following table summarizes the average concrete strength developed at the column face in each system as a function of the unconfined compressive strength of $0.85\, f_c'$ as recommended in ACI 318-71 [10] with the ϕ-factor taken to be 1.0.

Connection system	Average failure stress/$0.85 f_c'$	Number of specimens tested
(1) 1/4 in (6.35 mm) neoprene pad..	0.91	3
(2) 3/4 in (19 mm) grout bed.........	1.13	2
(3) 1/4 in (6.35 mm) neoprene pad covered by low-friction material...........................	1.20	2
(4) 1/4 in (6.35 mm) neoprene pad between 1/8 in (3.17 mm) steel plates.......................	1.32	5
(5) 1/4 in (6.35 mm) neoprene pad between 1/16 in (1.58 mm) steel plates.......................	1.02	2

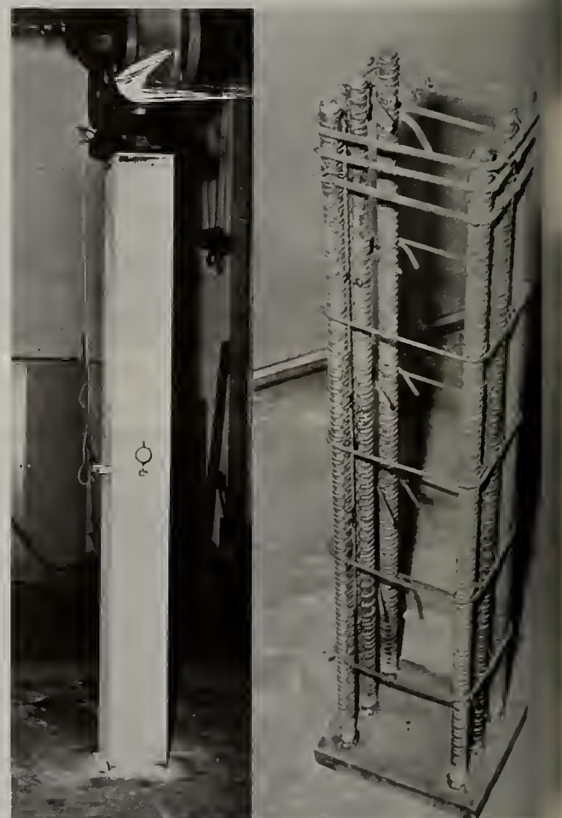

FIGURE 12. Specimen for connection test.

Test results on System (1) indicate that the bearing strength of the concrete was considerably reduced by the neoprene pad. A typical failure is illustrated in figure 13.

Results for System (2) indicate that with a grout bed some benefit is realized from the confining reinforcement loops. A typical failure is illustrated in figure 14.

Results for System (3) gives an indication of the order of magnitude of the effect of the neoprene (about 30 percent strength reduction), since the low friction interface limited transmission of the radially tangential shear forces from the neoprene to the concrete. The failure is shown in figure 15.

FIGURE 13. Failure of connection with 1/4 in neoprene pad.

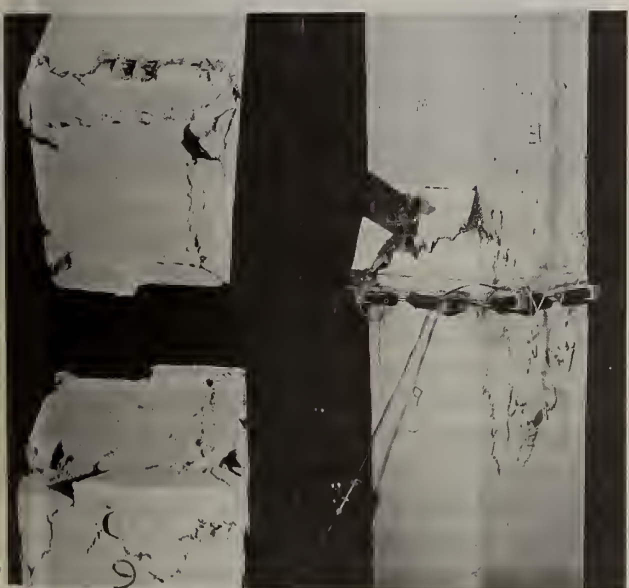

FIGURE 14. Failure of connection with 3/4 in grout bed.

FIGURE 15. Failure of connection with neoprene pads and low friction material.

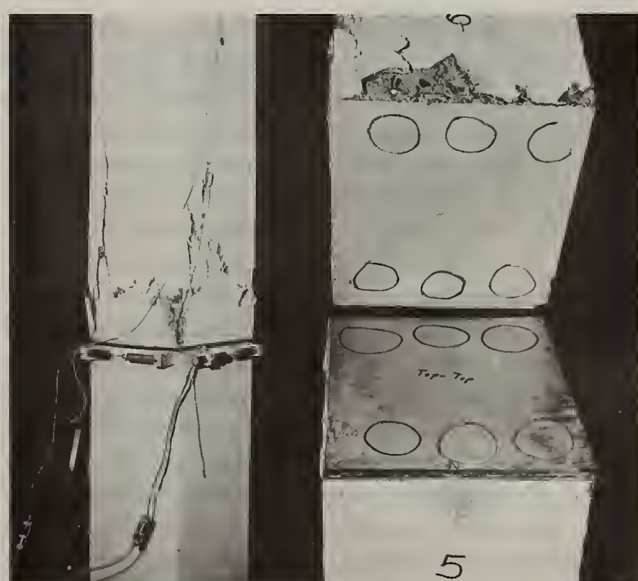

FIGURE 16. Failure of connection with steel-neoprene-steel sandwich.

Results for System (4) show very good performance. This is attributed to two factors:

(1) the steel plates resisted the radially tangential shear force exerted by the neoprene;
(2) the friction force between the steel and the concrete provided confinement additional to that provided by the reinforcement ties.

A typical test failure is shown in figure 16. Note that the main reinforcement bars left an imprint on the concrete face. The concrete face was molded, but did not crack. This is taken as an indication that the

FIGURE 17. Test specimen with grouted dowel.

FIGURE 18. Shear-test specimen before grouting.

concrete was triaxially stressed, and thereby developed high compressive strength.

Test results on System (5) indicate that the 1/16 in (1.58 mm) steel plate had insufficient strength to fully resist the tensile forces.

The above test results were consistent with results obtained under eccentric vertical load. On the basis of the tests it was concluded that System (4) could be used in critical bearing pads, namely those of the lower-story bearings. The concrete stress permitted under ultimate load will be $0.7 \times 1.1\, f'_c$ where 0.7 is a reduction accounting for strength variability and $1.3 \times 0.85 f'_c = 1.1 f'_c$ is the average strength achieved in the tests.

Another feature of the connection shown in figure 11(b) is the steel dowel. In some cases these dowels will be ordinary reinforcing bars, in others, post-tensioning strands. In most connections, the dowels are grouted after erection. However, in the two end walls, dowels are grouted progressively during erection. These progressively-grouted dowels will yield during erection, since the dowels alone do not have the load capacity to support the dead load. Yielding will proceed until the neoprene pad is compressed to the point where the load in the neoprene is equal to the applied load less the yield capacity of the dowel.

Several problems must be investigated:

(1) *What is the effect of the dowel, and of the yielding of dowels, on the compressive-load capacity of the column connection?*

This problem will be investigated by testing column connections with grouted dowels which will be pre-loaded to the full dead load before grouting, and comparing the results with test results from similar specimens which were not pre-loaded before grouting the dowels. One of these specimens is shown in figure 17. The dowel, the main reinforcing bars, as well as the reinforcing ties providing confinement to the concrete at the connection, are instrumented by strain gages. The instrumentation of the dowel will give an indication of the length over which the bond between the dowel and the grout breaks as a result of yielding. This will give some indication of the magnitude of the strain which occurs after yielding. The strain gages on the confining ties will give an indication of the contribution of these ties to the strength increase in the concrete at the connection.

(2) *What is the shear capacity of the connection, and will there be ductile behavior after a shear failure occurs?*

To investigate shear capacity and ductile behavior specimens will be subjected simultaneously to shear and vertical loads of various magnitudes. A shear test specimen being prepared for grouting of the dowel is shown in figure 18. These tests are considered critical, since the shear strength of the connection is necessary in resisting seismic forces as well as in providing the structural continuity required to prevent progressive collapse.

A third series of tests planned for this system will measure ductility available for resistance to seismic loading. In these tests, complete bents will be subjected to reverse cycles of lateral loading. First, several cycles of load will be applied, each large enough to cause a deformation of twice the yield deformation. Then, several additional load cycles causing a deformation up to five times the yield deformation will be applied. Finally, the frame will be laterally loaded to its full load capacity.

The previously discussed case is just one example of many performance evaluations now in progress at the National Bureau of Standards. The Bureau of Standards is carrying out these evaluations as the technical arm of the U.S. Department of Housing and Urban Development under the Operation Breakthrough program which has been discussed elsewhere [4]. It involves the construction of a number of selected industrialized housing systems on several demonstration sites. There were other cases of evaluation by testing where environmental conditioning had to be used, particularly in the case of structural adhesives. In addition to the present stage of performance evaluation, which precedes initial construction, the in-service performance of buildings constructed at the demonstration sites will be investigated. This investigation will be part of the overall performance evaluation, prior to volume production of the systems.

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16. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)

Considerable attention has recently been focused on the development of performance criteria. Performance Criteria are presently used in the U.S. by the Building Research Division of the Institute for Applied Technology, National Bureau of Standards, to evaluate innovative building systems.

While building codes and design standards are usually related to specific building materials and design solutions, performance criteria are derived from user requirements and are independent of specific technological solutions.

Many attributes inherent in traditional building systems which are acceptable to the user may not be present in untried innovative systems. The Performance Criteria therefore address themselves to many aspects of structural performance which are not considered in present codes and design standards.

The successful application of performance criteria depends on the feasibility of evaluating compliance.

Performance can be evaluated by analysis by judgement based on past experience, or by physical simulation. Deterioration with time must be considered, and performance criteria are viewed as minimum requirements which should be met at any time during the service life of a structure. An example is presented where physical testing was used to evaluate the performance of a high rise housing system.

17. KEY WORDS (Alphabetical order, separated by semicolons) Building; connections; housing; neoprene; performance criteria; performance evaluation; performance testing; reinforced concrete; standard tests; structure, testing.

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