CONNECTIONS

The EERI Oral History Series

Michael V. Pregnoff

John E. Rinne

Stanley Scott
Interviewer

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EERI also gratefully acknowledges partial funding of this project by the Federal Emergency Management Agency (FEMA).

The EERI Oral History Series

This is the third volume in *Connections: The EERI Oral History Series.* The Earthquake Engineering Research Institute initiated this series to preserve some of the rich history of those who have pioneered in the field of earthquake engineering and seismic design. The field of earthquake engineering has undergone significant, even revolutionary, changes since individuals first began thinking about how to design structures that would survive earthquakes.

The engineers who led in making these changes and shaped seismic design theory and practice have fascinating stories. *Connections: The EERI Oral History Series* is a vehicle for transmitting their impressions and experiences, their reflections on the events and individuals that influenced their thinking, their ideas and theories, and their recollections of the ways in which they went about solving problems that advanced the practice of earthquake engineering. These reminiscences are themselves a vital contribution to our understanding of the development of seismic design and earthquake hazard reduction. The Earthquake Engineering Research Institute is proud to have that story be told in *Connections.*

The oral history interviews on which *Connections* is based were initiated and are being carried out by Stanley Scott, formerly a research political scientist at the Institute of Governmental Studies at the University of California at Berkeley, who has himself for many years been active in and written on seismic safety policy and earthquake engineering. A member of the Earthquake Engineering Research Institute since 1973, Scott was a commissioner on the California State Seismic Safety Commission for 18 years, from 1975 to 1993. In 1990, Scott received the Alfred E. Alquist Award from the Earthquake Safety Foundation.

Recognizing the historical importance of the work that earthquake engineers and others have been doing, Scott began recording interviews in 1984. The wealth of information obtained from these interviews led him to consider initiating an oral history project on earthquake engineering and seismic safety policy. Oral history interviews involve an interviewee and interviewer in recorded conversational discussions of agreed-upon topics. After transcription, revision, and editing, the interviews and the tapes are placed in the Bancroft Library at the University of California at Berkeley for research purposes and scholarly use. Occasionally, interested professional organizations sponsor publication and wider distribution of interviews, as the Earthquake Engineering Research Institute is doing with *Connections.*

In due course, the Regional Oral History Office of the Bancroft Library approved such an oral history project on a continuing, but unfunded, basis. First undertaken while Scott was employed by the Institute of Governmental Studies, University of California at Berkeley, the effort has been continued on his own following his retirement in 1989. Modest funding for some expenses has been provided by the National Science Foundation.

Scott's initial effort has grown into an extensive program of interviews with earthquake engineers who have been particularly active in seismic safety policy and practice. Key members of the Earthquake Engineering Research Institute became interested in the project when asked to read and advise on the oral history transcripts.

The Earthquake Engineering Research Institute was established in 1949 as a membership organization to encourage research, investigate the effects of destructive earthquakes and the causes of building failures, and bring research scientists and practicing engineers together to solve challenging engineering problems through exchange of information, research results, and theories. In many ways, the development of seismic design is part of the history of EERI.

EERI Oral History Series

Henry J. Degenkolb	1994
John A. Blume	1994
Michael V. Pregnoff and John E. Rinne	1996

Interviews completed or nearing completion include:

George W. Housner
William W. Moore
William T. Wheeler
Robert E. Wallace

Interviews with several others are in progress.

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Michael V. Pregnoff

Foreword

In 1986 I conducted two oral history interviews with Michael Pregnoff, the first of about two hours' duration and the second a marathon day-long session. Both were held in Al Paquette's San Francisco office on Mission Street, not far from the Transbay Terminal. After an initial two-hour session, neither of us was s atisfied with the transcript, whereupon he proposed scheduling a full-day session. At first I was reluctant, since few oral history interviews run more than about two hours at the most. I agreed, however, when I saw that this was the way he preferred to do it. We met in Paquette's office for a second interview, which started at 10:00 in the morning and continued until about 4:30 in the afternoon.

It was not feasible to work from an organized outline, so we recorded verbatim a kind of stream-of-consciousness flow, interspersed here and there by my queries. This open-ended, nonstop style of interviewing seemed to work quite well. At midday we took a 45-minute break for sandwiches. At the end of the day, he did not seem to be at all tired, although I was definitely feeling some wear and tear. At age 86, and doing most of the talking, he maintained his energy level all day, and his memory was excellent. Although English is his second language (Russian is his native tongue), he expressed himself clearly on a wide range of topics with great fluency. Care has been taken to use Mike Pregnoff's original language and wording, virtually unchanged.

He focused mostly on what he considered to be "technical" engineering material and observations. When touching on more personal, human aspects of his experiences, he repeatedly apologized by saying, "The researchers don't want that," or "They won't need that..." Regretfully, there is relatively little here about his Russian origins. Although he talked a little about his family and early background, and about his experience in the revolution, he later requested that most of what was recorded be deleted from the transcript. In lieu of the personal material left out of the text, I include a few biographical highlights here.

Michael V. Pregnoff was born in 1900 near Vladivostok, Russia, and he received his engineering education at the Polytechnic Institute of Vladivostok. Caught up in the Russian Revolution and the brutal Civil War that followed, he soon decided to try his future elsewhere, making his way to San Francisco via Japan on a Japanese vessel routed through Hawaii, and arriving in August, 1922. He entered the U.S. with his Russian degree in engineering, but with very little English.

After arriving, the 23-year-old Mike Pregnoff worked for a relatively short time as a laborer in a brick factory, and also as a dishwasher, until his English had improved enough to give him the confidence to apply for a job at an engineering office. His first professional employment was with C.H. Snyder. He stayed with that firm until it became Hall & Pregnoff, established by the surviving partners of C.H. Snyder. The firm name later became Hall, Pregnoff and Matheu, then Pregnoff and Matheu, then PMB (Pregnoff, Matheu, and Beebe).

Among the innumerable projects Pregnoff worked on were the San Francisco Opera House; the Planetarium in San Francisco; Army and Navy buildings; the University of California's Cyclotron, Synchrotron, and Dwinelle Hall; the Hoover Library Tower and many other buildings at Stanford University . The listing of structures on which he or his firm worked fills many pages and includes sites throughout California, although concentrated in the Bay Area.

Always a very hard worker, he did manage to change his style after he reached his 50s, relying more on trusted colleagues. He spent time on professional engineering organizational activities and also made frequent visits to his cabin at Lake Tahoe, where he indulged his personal interests in nature study, hiking, and carpentry. At the time of the interviews in 1986, however, he still regularly participated in meetings of the Seismology Committee of the Structural Engineers Association of Northern California, and attended virtually every session as it worked on revisions for the Blue Book, the seismic design "bible" of the Structural Engineers Association of California.

Mike Pregnoff's recollections span the long years back to the early 1920s, with memories of some important early-day figures in earthquake engineering—the first generation, in fact. Three who notably influenced him in the early stages of his career were R.S. Chew, Fred Hall, and C.H. Snyder. Their examples and counsel helped instill a lifelong concern for quality engineering. Chew, in particular, taught him valuable lessons in earthquake-resistant design in the mid-1920s, at a time when few other California engineers were specifically designing for lateral forces other than wind. Many other prominent earthquake engineers from earlier times also figure in Pregnoff's recollections, including H.J. Brunnier, Gus Saph, Austin Earl, L.H. Nishkian, and E.L. Cope. Later, the older Pregnoff mentored the next generation. Below is an excerpt from unfinished oral history interviews with the late James Stratta, who early in his career worked for Pregnoff and Hall:

> *...one day with Pregnoff is worth one semester at the University. Mike Pregnoff is a very unselfish individual. He is extremely intelligent, extremely knowledgeable, extremely practical, and he liked to teach the people working for him to do things the way he liked to see them done. I guess all of us want to see things done our own way. He would quite often ask me to work overtime in the evenings... He would pay me time-and-a-half, which not all outfits did at that time. Then he'd take me out to dinner. Then afterward we'd come back, and instead of working on a project he would sit there and go over some of the basic fundamentals of engineering. It was actually an instruction period of how to do seismic design...*

In addition to their historical content, Mike Pregnoff's reflections have a wealth of astute observation on seismic design philosophy and engineering practice—both practice as it ought to be conducted and object lessons drawn from observations of unsatisfactory structural performance. The structural engineer's relations with the architect is another important recurring theme. He also emphasized the importance of distinguishing between appropriate and inappropriate uses of computers in structural design, and points out practical ways to avoid trouble by proper use of knowledge based on experience and application of pragmatic engineering judgment and intuition.

All in all, his explanations of the criteria and characteristics of good design and good engineering practice are convincing, clear, and expressed in language even a reader lacking engineering background can grasp. His oral history spells out his philosophy of good engineering practice. It should be a valuable resource for engineering and architectural students who wish to hear what one gifted old hand and dedicated engineer has distilled from a lifetime of experience as the essentials of practice.

Stanley Scott
Research Associate and
Research Political Scientist, Retired
Institute of Governmental Studies
University of California, Berkeley
January 1996

A Personal Introduction

This personal introduction should have been written by a structural engineer who worked with Michael Pregnoff—someone like Jim Stratta, Pete Kellam, or Bob Matheu. Unfortunately, they are no longer with us.

My first contact with Michael Pregnoff goes back about 50 years to around 1946, when I was returning from World War II. Like many other young structural engineers, I learned that finding work with established structural engineering offices depended on their work loads, which fluctuated. In my search, I called on the office of Hall and Pregnoff on Kearney Street in San Francisco. Their office consisted of an entry way, two or three small offices, and several drafting rooms. What impressed me as a job seeker was the large number of empty drafting tables.

The employment application form was a small 3-by-5 file card, on which I was asked to write my name, address, and phone number. When I talked to Mike Pregnoff, he was very courteous and explained that the firm did not have any work at that time. Like Gus Saph, H.J. Brunnier, and other prominent consulting engineers of the time, he took a personal interest in young engineers and encouraged them to stay in the structural engineering field, despite the difficulty in finding the right position on the first attempt.

Except for seeing Mike Pregnoff at meetings of the Structural Engineers Association of Northern California, my next contact with him was when I went to work in 1954 for the Oakland Unified School District as a structural engineer in their Department of Architecture and Engineering.

The school district was planning a bond issue to reconstruct or replace their pre-Field Act schools. Their planning was based on a report by the structural engineering firm Hall, Pregnoff, and Matheu, and the architectural firm Reynolds and Chamberlain, *Report on Structural Stability of Certain Old School Buildings in the Oakland Unified School Distrtict* (August 20, 1953). Mike Pregnoff was the principal author and had been personally involved with the background investigations for the report, as well as the structural calculations and cost estimates presented in it. This report was extremely innovative and forward looking, and much of its methodology is still in use today.

The report employed a system of building ratings—Good, Fair, Poor, and Very Poor—to describe expected building performance during future earthquakes with varying Modified Mercalli damage intensities. These ratings represented Pregnoff's judgment of the likely extent of damage, ranging from "negligible," "some," "considerable," "great," or "very great" life hazard for each school in earthquakes with modified Mercalli Intensities of VII, VIII, IX, and X.

This methodology was later adopted in the 1975 University of California Seismic Safety Policy, a policy still in effect today. For 20 years these ratings, when confirmed by more detailed seismic evaluations, have served as the basis on which the University has prioritized and funded seismic risk mitigation work.

Mike Pregnoff's fellow engineers knew him for innovative thinking and good judgment, and his advice on problems was often sought. When Robert Preece was a regional engineer for a large steel fabricator, he says he always found Michael Pregnoff easily approachable when he went by Pregnoff's office to discuss structural steel details. Pregnoff recognized the fabricator's strong preference for simple details that could be duplicated many times. Moreover, Pregnoff, ever on the lookout for ways to improve and simplify designs for economy, would also contact Preece for suggestions.

Michael Pregnoff led the way in establishing a committee of the Structural Engineers Association of California to study and recommend standards for evaluating drying-shrinkage properties of concrete studies. This early interest in drying shrinkage also carried over into his service as chairman of the American Concrete Institute's Committee on Deflection of Concrete Structures.

Michael Pregnoff's intuition, engineering acumen, and practicality shine through in this very special oral history.

Frank E. McClure
Consulting Structural Engineer
March 1996

Chapter 1

The Early Years

"I came to this country in August 1922, and in 1923 I started working for a man by the name C.H. Snyder."

Scott: To the extent possible, we like to make these interviews full-life portraits of the person being interviewed. After giving some biographical information, please discuss the development of the structural engineering profession in northern California, with particular emphasis on seismic safety design. I am particularly interested in your observations about the main events in that story, what was done, why it was done, who the key people were, what they did, and what you did.

Education in Russia

Pregnoff: Well, first thing, I was born in Russia in1900, and came here when I was 23. When I arrived in the U.S., I had just got my education in Vladivostok, Russia, and I was not an experienced engineer.

Scott: You had gotten your education in Russia. Let me backtrack on that a bit. How early in your youth did you know, or think, that you wanted to be an engineer?

Pregnoff: When I was about 10 years old, I loved nature. I would catch a wild bird, bring it home, feed it, tame it, and watch it grow. I would collect glass jars, and I'd make galvanic batteries out of them. I'd get 20 or 30 of them and I'd make a machine working by static electricity. I had a little lab at home.

Scott: This was when you were a kid?

Pregnoff: My father bought me a camera, one of those cameras that have glass plates instead of film rolls. I developed the plates myself, had a darkroom and everything. I made my own emulsion, as a new coat on a glass plate.

Scott: So at an early age you were interested in science and in experimenting?

Pregnoff: Yes. Then I got my education in Russia, and Russian education is quite theoretical.

Scott: Strong on mathematics, probably.

Pregnoff: Oh, yes. Mathematics was good for me, it helped me.

Scott: You got your education there, but when you came here you had no experience?

Pregnoff: Yes. For a couple months I was a dishwasher. Then I got a job in a brick factory. Finally I got a job with C.H. Snyder.

Scott: You did not practice in Russia at all?

Pregnoff: No, just got an education. By the way, our college was in Vladivostok and was temporary. It was formed by the professors who left central Russia to come to Far East Vladivostok. For a while it was not even Soviet territory—it was under a provisional government. So our college kind of emphasized fast and practical work. For instance, we would go into some factory and the professor would show us a boiler, as a problem. We would find how many shovels of coal they put in. One of us upstairs took the samples of smoke. Then we checked the temperature, and from this determined the efficiency of the boiler. It was a real job.

First Job in U.S.—A Brick Factory

Pregnoff: When I came here, I got my first job in Alameda, with Clark Company, making terra cottas and bricks for building construction. They put me in front of the large oven where they baked the bricks. There were several kinds of bricks, of different textures and so forth. When the baking is all done, they wanted to deliver the bricks and pile them in various piles, sorting out various textures. The fellow up in the cold oven takes three bricks and throws them down to me to catch. In front of me is a little cart, and I'm supposed to catch them and fill up that cart and roll it to a certain pile, put them there, and come back. He throws—you catch them, put them there. So then I was working like that—mostly it was with Mexicans, maybe Italians. They don't know where to put those bricks, so they ask me. That was the only thing about the job that was interesting, otherwise I never waited so much for a lunch time. It was three hours, and the same damn thing over again.

Then a big fellow, Fred—he was the superintendent in that plant—he looks at me and says, "Come here." I thought, "Gee, I'm doing something wrong; am I talking too much or what?" It's the second day I'm working, and he says, "Do like that; rub your palms. Look at that, blood, blood." I noticed—I am breaking my skin catching the rough bricks, so he gives me two pieces of rubber from a tire tube, with two slots for two fingers so you can catch the bricks against rubber. I noticed all those workers used them, too. This is the kind of men I met in U.S. To him I was a human being as well as a worker. I was a laborer for 30 cents an hour, in 1923.

When I had been working at the Clark Company more than a week, maybe it was the third day, Fred calls me again. I thought, "Boy, he's after me." He told me, "Those fellows don't know the distinction between those bricks." He told me to stand here and tell them to which pile to go. He just made a job for me.

Scott: You were traffic director for the brick handlers.

Pregnoff: He found it was worthwhile for me to get paid the 30 cents per hour and be more efficient. He was a good man, God bless this Fred. I'll never forget. He makes me think of the good things when I came to this country. Nobody ever, ever told me I was a foreigner. I became president of the Structural Engineers Association of Northern California in 1954. It's not that way in Russia or Europe. They're too nationalistic.

Starting Out With C.H. Snyder

Pregnoff: My practice starts from 1923. I came to this country in August 1922, and in 1923 I started working for a man by the name C.H. Snyder. He was a structural and civil engineer. Chris Snyder was a very good structural engineer. If you don't mind, I'll tell you about how I got the job with Snyder—do you want to hear that?

Scott: Sure.

Pregnoff: I came in to see him—I had prepared a little mechanical detail, as a sample of my drawing. The only job I could get was a draftsman's. Anyway I came to Snyder and showed him the drawing and he looked at it. He said, "You're crossing the 7s." I did not get the meaning of "crossing," and I said, "Please speak slowly, I do not understand English very well." He said, "Okay." He explained to me that here we do not draw a line through the vertical part of numeral 7. He talked to me for about 15 minutes, and he got the impression that I was poor or something. Then he looked at me and says, "Well, okay, I'll give you a job."

Then I started to walk out, and as I turned the door knob to leave, he called and said, "Come here." My heart sank. I thought that he was changing his mind. But he opened a drawer and took out a box of good instruments, German instruments. I knew what they were, although I did not have that kind. He says, "If you haven't got any instruments, you can use these." That's the kind of American I met. I never forgot that.

Scott: The draftsman job with Snyder came right after the brick-factory job?

Pregnoff: Yes, I couldn't get a draftsman's job right away. It took me about three or four months, so I could speak English a little better, and then I got the job with Snyder.

C.H. Snyder's Office: Learning Practical Engineering

Pregnoff: When I started to work with Snyder, they used me as a draftsman for a while. But I had been educated as an engineer, so I knew what I was doing. Very quickly Snyder gave me responsibility, and soon I became a structural designer. Snyder's office had good standards for concrete beams, with typical cross sections, footings. I studied those details carefully, copied them, and took them home and looked at them.

Scott: They had examples of typical details?

Pregnoff: Yes. Snyder had started it. They had a foundation detail, concrete details, veneer details, wood framing details, typical concrete block wall details, etc.

Scott: In other words, the office standards were demonstrated in these typical or standard details? In part I suppose this would be to save time, but it would also set the standards for the office's practices?

Pregnoff: Yes [shows book of standard details used in Al Paquette's office, where interviews were held]. Now here for instance, you have a detail of a concrete wall, and brick attached to it. They put it into the computer and the computer reproduces the detail on the drawing. The computer makes the drawings now. Here's a wood detail, a joist support at a stud wall. That's a truss joist with steel, here's how they hang the sheathing. There are all kinds of these details. Here's a steel beam and joist coming in.

Scott: That book is very voluminous and comprehensive.

Pregnoff: Yes. That is a collection of years of experience. Those details come from collecting details of past jobs. Come here. I'll show you something [points out computer-generated drafting being done in the next room]. Anyway, Snyder's office is the office where I learned my practical engineering.

Scott: That goes back to 1923?

Pregnoff: Yes, to 1923. Before opening his office, C.H. Snyder worked as a sales engineer for Milliken Steel Company, and in those days, years ago, he would come to the architect and say, "I will give you the layout and structural steel sizes and you will give the steel contract to my company." That's the way they did it in those days.

Then in about 1910 Snyder opened his own office. He specialized in steel, concrete too, but mostly steel. He made wonderful drawings. Every contractor, when they got his drawings, they wanted to build his jobs. It was the same thing with us in my firm's practice. Lots of contractors liked our jobs because they were well detailed.

Scott: That made them easier to follow.

Pregnoff: Yes, not much uncertainty. The most critical time for a job is when the estimator estimates the job. They give him maybe three or four weeks, or maybe only two weeks. If you have the drawings more complete and definite, there is no guesswork and the job can be estimated cheaper.

F.F. Hall, Snyder's Chief Engineer

Pregnoff: I learned a lot of things from Fred [F.F.] Hall, Snyder's chief engineer. He designed City Hall, years ago.

Scott: Fred Hall designed City Hall?

Pregnoff: That's right. He was chief engineer in Snyder's office, and was one of the main engineers I learned from. I remember, for example, when the roof collapsed during construction on one of our jobs in Berkeley. A telephone call came, telling us about the roof collapse. A portion of the roof had sagged—it really did not collapse, didn't kill anybody, but sagged and was distorted. So right away somebody in the office said, "Let's look at the drawings, to see who made a mistake." Everybody started looking around and asking—"Who did that?"

Fred Hall says, "What the hell is the matter with you fellows? Who cares who? What I want to know is how are we going to fix it? Think how we're going to fix it, to repair it a soon as possible. Don't think about who did it, we don't care." That's a good attitude. He was a very good man, and he was one of those men who did some designing himself. He liked his engineering. For instance, he would come into the office early, and be working when I came into the office and would say, "Good morning Mr. Hall," but he would not say, "Hello." I understood why. He was just so much absorbed—he was concentrating. Hall and Snyder were good at details. I was fortunate to be working for that office. They wanted concrete members large so there was a lot of room for reinforcement, a lot of room for concrete to flow. Some other engineers, following the code, make them shallower, which architects like.

Scott: You're talking now about columns?

Pregnoff: Beams and columns. Make them large, with plenty of room for concrete. Naturally the larger, the deeper the beam, the less demand for the reinforcement. Theoretically, the shallow beam having more steel is better, because it is the steel that provides ductility. Concrete is not ductile. So theoretically it is better to have the beam shallower and with a lot of reinforcement. But practically, it is better to make it deeper and have less reinforcement.

That's what I learned from them. The size of a member is decided by the span. If a beam is 20 feet long, it will be 20 inches deep; if 14 feet long, 14 inches deep. But some engineers, with a 20-foot long beam will make it only 16 inches deep. I really think that I am more or less one of the conservative engineers. I learned it from Snyder and Hall.

I guess I was very fortunate that I learned all my practical engineering from a very fine office, which had a good practice. I learned from Snyder's office, which was one of the best. If I had gotten in with some other young engineer, I would never be what I am, but fortunately I got in with Snyder. In San Francisco there were maybe five designers like Snyder, doing all the big buildings. These were Henry Brunnier, C.H. Snyder, L.H. Nishkian, E.L. Cope, and Austin Earl. There were also engineers like [R.S.] Chew and Saph. Gus Saph was in a single-man office, and he was also a gentleman engineer. I would sit and talk with him, and learned a lot from him. Those older men like Chew, Saph, Cope, Fred Hall and Snyder were peers [of each other] back when I was only starting. I started at 23, so I was a young man at that time.

Scott: They would have been in their 40s or so when you were in your 20s.

Pregnoff: Now I am older [86 years old at time of interview in 1986], and very few men like me are left. I don't even know who they are.

Seismic Design—Influence of R.S. Chew

Pregnoff: In those days, however, when I started to work for Snyder, we did not design for earthquakes at all. The exception was when Chris Snyder was engaged in designing the present Opera House, about 1928. At that time there was another engineer in private practice, in a one-man engineering office, who would do some of Snyder's work. His name was R.S.

Chew. I think he was an Englishman. He was quite a gentleman, a real good engineer, and a researcher, too.

Occasionally, Snyder would send me to Chew, who would do some part of Snyder's work. When Chris's office was loaded, he would give Chew part of the work, so I would go to R.S. Chew's office and work with him—do some drafting and computation. He instilled in me a desire to study the effects of earthquakes on buildings.

I had gotten acquainted with Chew at the time of the Tokyo earthquake in 1923. Chew and I talked about it a lot, and I learned a lot from him. Many engineers thought, like Snyder, "If a building is designed for 30 or 50 pounds per square foot of resistance to wind, it's good enough." In those days also, engineers like Snyder thought the exterior concrete walls around the building or brick wall, called curtain walls, were there to keep the weather out and they were not considered as resisting lateral forces due to quake at all.

Then, in 1928, when Snyder's office was designing the Opera House, Chew was engaged by Snyder to run the job, and I was Chew's assistant. Even back at that time, on that job we tried to design the structure for 10 percent of gravity horizontal force. Snyder had never done that sort of thing himself, but Chew did on the Opera House, which was a structural steel building. Chew said, "We will put a horizontal force of 10 percent of gravity in our computations and see how much effect that will have on our sizes [of columns and beams].

[Later, in 1939] I designed the 17-story Tower of the Hoover Library at Stanford University. At that time there were no local earthquake codes, but as I said, I was educated by R.S. Chew, and so designed the building to resist earthquakes. It is a structural steel frame building, and I used diagonal bracing to resist lateral forces.

Damping and Brick Walls

Pregnoff: Chew also wrote a book on earthquake design.[1] He believed that buildings should be flexible. He also believed that structural steel frame buildings with exterior reinforced brick walls were the best for earthquake construction. Better than concrete walls, because brick walls have the ability to give and work together with the steel—we call it damping. Damping is a very important factor in an earthquake-resistant building, in addition to the structural resistance and the force resisted by the columns. It dampens the energy that the building receives from an earthquake. Damping can save a building.

Chew considered a bare steel building the worst, because it may synchronize with the earth motion and wobble more and more. All the nonstructural elements—like regular nonstructural partitions—produce quite a bit of resistance to earthquakes. Even if they are failing, they're also working at their utmost and they provide damping. Some of those older tall buildings in San Francisco did not collapse, but withstood the 1906 earthquake, and then were damaged mostly by fire. The nonstructural elements helped them to resist earthquake forces.

1. Chew, R.S., *An Approximate Measure of Earthquake Effect on Framed Structures.* Self-published by Richard Sanders Chew, San Francisco, CA, 1933.

Chew believed brick walls to be best because they absorb the energy of the motion. Concrete walls, being stiff, do not yield the way brick walls do.

Scott: Brick walls yield along the joints?

Pregnoff: Yes. There is mortar between the bricks, and each layer of mortar cracks a bit and there's more play. In San Francisco a lot of tall steel buildings had brick exterior walls, and stone exterior, too.

Scott: It was the *combination* of brick and steel that Chew said was good?

Pregnoff: Yes.

Scott: Was Chew the principal person who instilled in you the conviction that earthquake design ought to be done?

Pregnoff: Yes, he was one of the first persons who did. I was a young engineer, 28 years old, having been born in Russia in 1900. Chew instilled in me the desire to study the vibration of buildings during earthquake motion. This is quite a complicated problem, a very complicated problem. And Chew instilled in me the desire to design buildings for earthquakes.

Scott: Once when interviewing Henry Degenkolb, he got out Freeman's old book[2] and opened it to a composite panoramic photograph of the city burning in 1906. One of the panels showed a group of people on top of the Fairmont Hotel observing the progress of the fire, which was then still mostly in the eastern and central part of town. Henry pointed to one person standing off sort of to himself, and said, "That's R.S. Chew."

1906 San Francisco Earthquake Photograph

Pregnoff: I have that picture right here in my copy of the book. The photo is titled "The Burning City." Many times in my practice, I have been asked to evaluate an existing tall building in San Francisco. Can we use the building? Can we remodel it and use it, or is it dangerous? How will it behave? I always showed this picture to people; it shows tall buildings still standing. They do not meet present-day code requirements for earthquake-resistant construction, but they have an inherent strength. Our observation shows that buildings of this type—structural steel frame, with bending moment resisting connections in all joints—give a good account, as it shows in the picture.

The buildings in question are still being used in San Francisco. The Call Building, Mutual Bank Building, Mills Building—all of them are still used. Some of them were burned out, like the Call Building, as it was called at that time, at the corner of Third and Market. In some of them, like the Palace Hotel, the steel columns got so hot that they bent and buckled. Later they were straightened out. The Sheraton-Palace Hotel is still being used.

This is a good picture. It shows that we shouldn't be too panicky, saying everything is going to collapse. Of course, the unreinforced brick buildings, of which there are a lot in San Francisco, are very dangerous.

Some of the mortar in those unreinforced masonry buildings is so poor that you can get

2. Freeman, John R., *Earthquake Damage and Earthquake Insurance.* New York: McGraw-Hill, 1932.

it out with a pencil. Sometimes you can dig it out with your thumbnail. It is more sand than cement. They say that such mortar is only trying to keep the bricks apart, instead of sticking them together.

Scott: But Chew emphasized the combination of bricks, and good mortar, of course, and reinforcement. Because with reinforcement, if the brick part starts to give a little bit too much, it's held back by the reinforcement.

More About R.S. Chew

Scott: Can you tell me a little bit more about R.S. Chew? Was he indeed a practicing engineer in that picture from the 1906 earthquake. How old a fellow was he in the '20s when you worked with him?

Pregnoff: When I worked with him [beginning in 1928], I suppose he was about 50 years old, and I was 28. [Gus] Saph was about Chew's age.

Scott: So he was quite a young man when he was on top of the Fairmont in 1906.

Pregnoff: I don't know why it's of so much interest to you. What the hell difference does it make? He was older than me. He was a peer of Gus Saph. Those people, like Chew and Gus Saph, didn't want to open large offices. They just hired one draftsman. They were satisfied with that and they did a good job.

Scott: Chew and Saph both operated pretty much as one-engineer firms?

Pregnoff: Yes. Chew had certain clients that were with him who were happy. He did high-class work. The telephone company was giving him jobs. He did what was right. He did it intuitively. And he had his own theories. He wrote a book and published it, but he didn't sell it. Typewritten, on 8-1/2-by-11-inch pages. I have two copies, a first edition and second edition. If you don't care which one, I'll give it to you.

Scott: I will borrow one and photocopy it.[3]

Pregnoff: I observed something else about R.S. Chew when we were doing the Opera House in 1928. We had one Swedish engineer with whom I disagreed on some details—I didn't like the way he had done them. I was Chew's assistant, so I went to Chew and said, "Look at how he is doing this—he's wrong." Without even looking at the details, Chew said, "You should say, I think he's wrong." That's a gentleman for you. You don't say, "I want you to do that." You don't speak that way to the people you are working with. Instead you say, "Let's do that." A lot of us do not compliment a person for the good things he's doing, but when an error comes in, we give him hell. We should remember that each one of us has more good points than bad.

Few Designed for Earthquake Resistance

Scott: While you are on this topic, let me ask a couple more questions about early seismic design or lack of it. You indicated that after the 1923 Tokyo earthquake there were discussions about earthquake design among the San Francisco engineers.

Pregnoff: Yes. After the 1923 Tokyo earthquake, San Francisco structural engineers at their meetings discussed problems of the

3. Chew put out three editions of his self-published book—1933, 1938, and 1944.

design of buildings to resist earthquakes. They talked about seismic design, but only a few, like R.S. Chew practiced it. Chew designed buildings for the telephone company. He was their favorite engineer. They gave him jobs. Henry Degenkolb, a San Francisco engineer, said recently that while they were remodeling some building that R.S. Chew had done, they saw in his details that he had designed for earthquakes. He tied all his buildings together, thinking of earthquake forces.

Like Hank Brunnier said, "Make them act as a unity." If the buildings in Mexico [in 1985] had been tied together with large steel bars in concrete beams, how could you pull them apart? In the photos you can see beams at columns that got separated and fell to the ground. The floors pancaked, one floor on top of another.

Scott: Was design for earthquake resistance mainly a matter of the judgment and practice of the individual structural engineer?

Pregnoff: Very few engineers designed for earthquakes. In those days, there weren't so many engineers in San Francisco. The good offices, maybe five or six of them, were run by men with judgment. They tied their buildings together well. They would not design unreinforced brick buildings of 3-4-5 stories high. They used steel frames.

I cannot elaborate freely on the prevalence or nonprevalance of seismic design practice in the '20s. In 1925 when I was working for Snyder, Chew showed me in his office how he computed lateral forces due to earthquakes in the buildings he designed. I know that Snyder designed buildings for wind forces only. No one, except Chew, ever told me that he computed forces due to quakes. Because engineers discussed earthquake problems after the great 1923 quake in Japan, however, I feel that perhaps a minority of them in San Francisco computed lateral forces induced by earthquakes in the buildings they designed.

Tying Buildings Together

Pregnoff: San Francisco did not adopt an earthquake code until 1948. Thus, there was no earthquake code in San Francisco until 1948. Los Angeles started to design for earthquakes before San Francisco. I guess [in San Francisco] there was too much influence of the builders, or somebody. They just don't want to spend money on earthquake design. Anyway, Los Angeles was the first one to start.

Scott: I gather that engineers used some rules of thumb to guide them in trying to design for earthquake resistance?

Pregnoff: Yes. I mentioned Henry Brunnier, a prominent San Francisco engineer. He used to say that the most important thing is to tie buildings together so that each acts as a unity in one direction, and also in the other direction. I think that was what some buildings in the recent Mexico City earthquake lacked. If you look at the pictures of the damage by the Mexican earthquake, you can see the beams just separated from the columns and collapsed. Some people say that in Mexico the soil caused large vibrations, but also their design probably wasn't as good or as carefully detailed as ours.

Scott: In terms of tying the structure together?

Pregnoff: Yes. Also, their concrete probably wasn't as good as ours. We control our concrete better than they do. We have inspectors.

Chapter 2

Observations Based on Practice

"If you do something that is good structurally, it's good aesthetically, because forces are flowing."

Pregnoff: I had a pretty good life, and I was pretty lucky. I am now 86 years old, so I don't regret it if I die. I got so much enjoyment in life, it never refused anything. I think structural engineering is a good profession if you are more or less good, but if you're not, it's no good no matter what the profession is.

Architect Has the Say

Pregnoff: If I had a son now, I would like him to be an engineer, but not in [structural design and] buildings, like me. That is because we work always with architects. When you work with architects, as I told you, the fate of a building is decided by the architect as far as earthquake is concerned. Some of them make the design complicated—some of the them make the buildings round or octagonal, or cut out some portions, or do all kinds of things. You [as engineer] have to adapt yourself to those situations. After all, people don't see your engineering, but they do see their architecture. Some of the round buildings, octagonal buildings, etc., look good on the outside.

But [now] I'd like to be a sanitary engineer or highway engineer—then you are independent entirely. There's a mechanical engineer in the building, [but] of course he works with architects.

Scott: Your concern is that the architect tends to have the ultimate say in building design?

Pregnoff: Absolutely. He has to, because he gets the job from the owner. The owner doesn't go to me, he goes to an architect. The owner doesn't pay us money, he pays the architect and the architect pays us. Some architects were a little stingy in paying us our fee, trying to get more for themselves. You had to argue with them a little bit.

Other architects, high-class architects, big architects, too, were our clients. Like Tim Flueger, Arthur Brown [Bakewell & Brown], then Stone, Marichini, Patterson. Some architects argue, but Ed Stone doesn't argue. Rex Allen knew more about hospitals than doctors. He was our client; we did a lot of work for him.

Scott: Ed Stone was one of those who were different?

Pregnoff: Yes. He was high class. He gets a fee of 8 or 10 percent—smaller architects get 6 percent. Nowadays, sometimes the mechanical cost is 40 percent of the total cost of the project, and 20 percent is structural, adding up to 60 percent. So the architect is doing 40 percent or even less of the total. Architectural costs are slightly less than mechanical.

Dealing With Architects

Scott: Would you discuss the issue of relations with architects a little more. From what you just said, it is clear that you see it as a very important matter.

Pregnoff: Yes. The first earthquake regulations in San Francisco were enacted in 1948. The architects are not sympathetic to earthquake problems—it interferes with their planning. No engineer can make irregular buildings behave properly during an earthquake. And architects don't want seismic joints as separations. But when you face that [design] problem with an architect, you have to do the best you can. You have to give them the impression that it is impossible for the engineer to do everything.

The fate of the building during an earthquake is decided by the architect. Good architects engage the engineer and have faith in him. They ask the engineer to develop a sound scheme, instead of giving him a drawing of their layout and expecting him to adapt to it. The good engineer is the one who can say no to the architect.

The bigger the architect, the easier he is to deal with. The smaller the architect, the less easy, because he is not as imaginative. When we had a famous New York architect, Ed Stone, anything I told him was acceptable, because he had imagination. We designed the Stanford University Hospital for him—a big building—and anything we asked for, he said, "Okay, Mike, go ahead."

We designed a large Pasadena vitamin pill factory for him. He asked me to plan that building. I planned it in concrete, flat-slab construction, the cheapest. A concrete beams-and-girders scheme is a little more expensive, and steel frame still more expensive.

Scott: These were three alternative building types?

Pregnoff: That's right. I was supposed to give him the building schemes, indicating which I would recommend. We had a meeting with Ed Stone in his office, my partner Matheu, and two or three of his men. I presented three schemes. Ed asked, "Mike, which is the best scheme?" I said, "The steel." "Let's use the steel," was his decision.

Concrete Arches: Combining Architecture and Engineering

Pregnoff: We also worked with Ed Stone on the Perpetual Savings Building in Los Angeles. He wanted the exterior wall to be with continuous concrete arches, and asked, "What kind of arches should they be? What is the best for you?" My partner, Bob Matheu, went to the blackboard and drew a freehand arch saying, "This is structurally the best shape." Ed said, "Okay, that's the way we're going to make it." That was one of the first concrete arches we did with reusable moving plastic forms. It's a 9-story structure, quite large.

Scott: I take it the arch was a key architectural feature?

Pregnoff: The arch was architectural, but at same time also structural. Bob Matheu drew it. And Stone said, "It looks good that way."

Scott: Was Matheu's initial drawing of the arch a matter of his aesthetic intuition, or intuition based on the engineering and structural role of the arch?

Pregnoff: Engineering. Take a Greek or Roman column; they were good structurally and also architecturally. If you do something that is good structurally, it's good aesthetically, because forces are flowing. To me it's a very good-looking structure, the way Bob Matheu drew it.

Scott: So the two—architecture and engineering—came to focus very well here. But Matheu the engineer originally drew the arch, rather than Stone the architect.

Pregnoff: Yes. He drew the arch in our office to a large scale. They scaled from that, and we analyzed it for earthquake forces.

Colleagues

Jim Stratta.: Learning the Ropes

Scott: As you discuss your practice, I want to ask you about Jim Stratta, especially because I think you were one of the key people he learned the ropes from. Frank McClure told me that he thought Jim learned his earthquake engineering from you. Did Jim learn a lot about seismic design when he worked with you?

Pregnoff: All engineering, not only earthquake engineering. Everything.

He worked for me, and then he [Stratta] and Al Simpson left and opened their office. Occasionally they would call me up and ask me something—for a little consultation. Once, Stratta called me and said, "Mike, let's go to lunch." I said to myself, "They have another question." So when we started to eat, I said, "What have you to say?" He said, "We two [Simpson and Stratta] started to talk to each other, and we thought what lucky guys we were, working for you. That's how much we learned from you, so we asked you to have lunch, and now we're telling you."

To me, it was worth more than money. More than money. Because people appreciate. I never tell people that they are working for me. I work with them, together. They're working with me.

Graham and Kellam; Paquette and Associates

Pregnoff: Pete Kellam also worked for us, he's at Graham and Kellam now. Graham inherited his office from [William] Adrian—who was a big engineer. This office of Paquette and Associates, where we are now talking, was [Frederick] Kellberg's. He designed the Cow Palace, a tremendous big building, and a very original design. Paquette's office was inherited from him [Kellberg]. Mr. Paquette is more like me, he's a little younger than me, maybe by four years—we're of the same vintage.

Concern About Shift Toward Bigger Offices

Pregnoff: There is a tendency now to build up big offices, where owners more or less act as businessmen, and hire the capable men to do the engineering. This is not as good as it used to be with smaller offices.

Scott: Is your concern about size in part because the men at the upper level—the principals—can't really practice engineering, that they have to be engaged in administration or office management? What about the engineering practice of the other 190 people in the 200-member office?

Pregnoff: Those offices also engage special promotion men who solicit jobs for them. They subscribe to special magazines to give them leads ahead of time—somebody is planning all the time to get new jobs.

Our Style: Materials, Workmanship, and Inspection

Pregnoff: The Pregnoff and Matheu style of working was first we received the project from the architect. Then Bob Matheu and I would sit down, asking, "How are we going to do it?" The choice of materials is very important for the resistance of a building to a quake. Suppose you made it out of brittle material. Naturally, it will behave differently than if there's a more flexible material to absorb the motion and absorb the energy. We try to choose proper materials. Unreinforced brick, of course, is not good, and they won't allow you to build with it.

Even with reinforced brick, however, workmanship is very important when you're putting all the little pieces together. It is very important how you place that mortar, how you fill in all those joints. A block wall is a wall of hollow concrete blocks. Every cell is filled with concrete grout and a bar is inserted in it. And the bars ought to lap properly. Workmanship is very important. They pour three feet at a time and tamp, and do that again and again.

Sometimes they pour from the top—the hole is only 4 inches by 4 inches, but they'll pour 10 feet. We had an occasion on one of our jobs, involving a 1-story school with reinforced block walls. Every block is supposed to be filled with concrete grout, so it will be good for earthquake forces. Two or three years later the school decided to make an addition, and they wanted a door in the wall.

When the wall was cut for the door, they found the bars were there, but no grout. Because the pouring was not careful and was very erratic, they didn't fill in the all of the wall voids. All walls were tapped with a hammer to determine which areas had voids. Many areas were uncovered. They had to drill holes in the shells of blocks and pump the grout into those voids. The school district paid maybe $30,000 for that. They could have sued us, but they did not. Inspection was bad; I don't want to say that the contractors willfully did that.

Scott: This is something that the contractor would have been supposed to handle?

Pregnoff: That's correct.

Scott: Also, if it was a school, wouldn't it have been under Field Act inspection requirements?

Pregnoff: Yeah, it was inspected, but when they're pouring you aren't going to stand checking item by item. It's a continuous inspection—the inspector is walking around everywhere. But you just never know everything that happens.

An uncle of my partner, a general contractor, was building many Stanford University structures. He built a block wall 60 feet long and 16 feet high. It stood there for quite a while, and then they also decided to make a door in it. When they cut for the door, there was reinforcement but no grout.

They started investigating. You know what the subcontractor had done? At the top of the wall he put a newspaper down for a foot below, then just poured the top. A crook. A general contractor does not pour walls, he gets a subcontractor, a concrete man who will mix and pour that concrete grout. The uncle had his own superintendent, who was walking around. Now, that man was a good contractor, and yet here this occurred with a good contractor. Crooks were there and the superintendent didn't see them put a newspaper in to stop the concrete grout.

Scott: They stopped it so the concrete would not go on down, so they saved themselves a lot of concrete?

Pregnoff: Yes. The uncle refilled the wall at no cost to the university.

Important: Inspection During Construction

Scott: So it is important that a construction job be inspected regularly while it is in process?

Pregnoff: Yes. Inspection is very important. In our office at certain times we had 65 projects under construction and in the design stage at one time. We had our own inspectors, and the inspectors were going around to all the jobs, all the time. We had three inspectors. We bought them cars. They drove in our cars and inspected the jobs.

When a job starts, we have a construction meeting. We give the contractor two letters. One letter says, "Our inspector has no right to change anything shown in the drawing. Any changes should be made in writing—in a request made to us and answered in writing." A second letter stated some particulars—no concrete should be poured before our inspector has inspected it, etc.

Some of the architects in our agreements said that during construction the engineer should visit the job at least every two weeks and report in writing if, in general, the job is built according to drawings. We visited oftener—our man just drove around all the time. They can just glance and immediately see whether the contractor is doing right. The contractor would throw things around, ride across the bars, bend them, do all kinds of things, fail to clean them. Rain, foundation filled with water.

Scott: So it was your practice to do inspection?

Pregnoff: Yes. That is very important.

Scott: Did you do that from way back at the beginning?

Pregnoff: Yes. As long as I practiced. We inspected to protect ourselves, and of course we're protecting the owner also. I think the main problem is a lack of inspection. We asked our engineers to go Sundays and look at the jobs they designed. They would charge us for it—that's their time. If you are going to interview Stratta, ask him, "Do engineers go and look at their jobs?"

Some engineers don't do that. It costs money. We were fortunate that our fee was high because we had large jobs, and big architects. When the architect is smaller, and on smaller jobs, he charges less for his work and the engineer gets less money. The end result is they can't afford inspection. They don't even look at the jobs unless the contractors don't understand the drawings and call the engineers. Then they'll answer. Otherwise the engineers don't go there to the jobs.

Scott: They don't even visit the job sites?

Pregnoff: Oh, no. You'd be surprised what the actual practices are. Say you have a big job. It's a concrete building, and all bars should be already in place in the entire area. Nevertheless, they're pouring, yet they have not finished placing the bars in certain spots. Rush develops on every job. Rush develops immediately, and then they re rushing to do everything. On account of rush, the work is not good.

Scott: The contractors have deadlines, and also they're trying to save on employee costs.

Pregnoff: In Russia they have a rhyme, that translated goes something like: "If you rush, people are laughing at you. If you rush, you make a fool out of yourself." What is similar to that in English?

Scott: One rough approximation is "haste makes waste."

Pregnoff: After every earthquake—even the recent one in Mexico [1985]—it is always said that the buildings were not built according to drawings. In some buildings the reinforcing bars, instead of being welded, were tied together with wires.

Scott: They departed from the drawings?

Pregnoff: Yes. They departed from the drawings.

A World War II Recollection: Designing Concrete Ships

Pregnoff: During World War II, the firm Ellison and King, Structural Engineers, San Francisco, were commissioned to design concrete ships, and I was engaged by them to be in charge of structural design. The ships had no propelling power. Several of them were built in San Francisco and towed to the Caribbean Sea to get the bauxite ore. The ships were 300 feet-long, and divided by bulkheads into 30-foot compartments [holds].

Fundamentally, ships are designed to be capable during storms of spanning between crests of waves 300 feet apart, which is the length of the ship. Also it must be capable of cantilevering 150 feet on each side of its midpoint when being lifted by a wave. Ships are also designed to resist water pressure for different conditions of loading and unloading. Its interior partitions [bulkheads] are designed to resist the pressure from the water. That way, if the ship is torpedoed, it is designed so that it will not sink.

In those days we had no computer programs. We solved mathematical equations by using slide rules and electric calculators. When we start to use theoretical equations we have to assume trial sizes of all continuous members. For this task I used Ira Kessy. He was able to compute the preliminary sizes which were close, on the safe side, to the finally computed sizes. I do not know his educational background; we never talked about it.

Somehow, he was breaking complicated problems into simple elements of structural mechanics. Apparently, he was free of the mathematical straitjacket, which could otherwise blind him and prevent him from using his great insight, perception, and intuition. Some engineers develop intuition, judgment; they even don't know why they do it that way. Just the way Bob Matheu drew the curves for the Perpetual Savings Building I mentioned.

Scott: How do you develop that ability?

Pregnoff: You're born with it, and you develop it more by using it. Some engineers [on the other hand]—and I have had them in my office—were good at mathematics when they were in high school or college. They thought they would be good in engineering too, so they decided to be engineers. But they're not engineers, they're mathematicians. When it comes to computations, one of them can make computations, but he cannot develop workable details, while another fellow who is not as good in computation is good at details.

Shifts in Work Patterns

[*Editor's Note:* Over the years, Pregnoff's practice and work patterns underwent significant changes, which are outlined briefly here. He was a very hard worker, but after reaching his 50s, he looked to reliable colleagues for more of his firm's work, spending additional time on engineering association activities. He also began making frequent trips to Tahoe, where he enjoyed his cabin and could pursue such interests as nature study, hiking, and carpentry. His opening comment that follows relates to a period, probably shortly after World War II, when reliance on trusted colleagues had enabled him to modify his long-time pattern of long hours devoted to office work. Later, with the death of his senior partner, Hall, and departure of some key employees, things shifted again in ways he describes below.]

Scott: You told me that earlier you typically maintained a heavy schedule, working nights and so on. When was that?

Pregnoff: Before Bob Matheu joined us, when I was with Fred Hall. When I designed the PG&E building, for example. Many jobs I designed all by myself. I loved to work. [Then] Fred and Bob told me, "Mike, you work hard; go to your cabin." They wanted me to go, so I did. So in the summertime I just went up to Tahoe every week and stayed there for three days. After that I would only watch how everything was going on.

Later Fred Hall, my partner, died and I and Matheu ran the jobs. We had good men like Stratta and Simpson. They had reached a stage that we didn't have to worry. They knew our style, too. When our office got a new job then, I would work only in the preliminary stage.

In starting a new job in our office, Matheu and I would determine the steel, what centers the columns are, etc. We would determine preliminary sizes. We know the people who estimate the job quickly, we would tell the architect how much it will cost. Then we would give it to Jim Stratta, our engineer, to carry out the scheme. He tells the draftsman to lay it out, and he makes formal computations. Finally, they make drawings and specifications.

Later Simpson and Stratta left and opened their own business. Then, years later, Simpson lost his life in a fire that occurred in the Yacht Club in San Francisco. That was too bad. Jim Stratta is now retired. He's doing consulting engineering on electronic buildings—he's expert at that.

In about 1945, when the war ended, the name of our firm was Hall and Pregnoff. Matheu had started to work for us in 1942, but he went to war. He came back when the war ended. Later he was admitted into partnership, and the name of the firm became Hall, Pregnoff and Matheu. When Fred Hall died in 1955, the name of the firm became Pregnoff and Matheu. Now Matheu runs the firm Pregnoff and Matheu in Palo Alto. I am no longer a partner. I am doing some consulting work on my own.

In about 1970 Ken Beebe, our partner, started to take the jobs of designing the large ocean oil pumping platforms and towers. The oil business needed a lot of towers, and our firm was designing them as a major business. Eventually we formed the PMB Corporation. I like to design buildings and not oil towers. I sold out my shares. So did my partner Bob Matheu. Later, demand for oil towers disappeared. PMB corporation was taken over by the Bechtel Corporation. Still later Beebe—practically retired—is maintaining the job of director in the corporation.

Chairman of ACI Committee

Pregnoff: For eight years, I was chairman of the Deflection of Concrete Structures Committee for the American Concrete Institute (ACI). We produced a report, practically a book. I worked hard at it. I did not spend much time on our business then.

Scott: When would that have been?

Pregnoff: Starting in 1956 or 1957. Then Branson took the committee chairmanship over. I'm still a member of that committee, but I'm not chairman now.

Designing: Intuition and Judgment

Pregnoff: We did a lot of reconstruction of Stanford University buildings. The Stanford University buildings have the sandstone exterior walls. They didn't want to lose them during construction, so when we were taking the inside works out, we temporarily supported the free-standing walls so they won't collapse during a quake at construction time. We reconstructed maybe six or seven buildings at Stanford. This was seismic retrofitting. Then later some other engineers reconstructed other Stanford buildings. I remember one firm saying, "We followed Pregnoff and Matheu's style." They did it our way because they couldn't do it any better way. They gave us a compliment.

That's why I said a building should not necessarily be designed by the computations. After computations are made, you have to look them over and see if they give you reasonable sizes. In planning a job, I determine sizes first, with my preliminary computations, and if the final computation does not quite agree, I make it a little bit larger or maybe a little bit smaller. That's planning the job.

Look at the old-timers—the Greeks and those other old-timers of many years ago. There were no computers then, there were no slide rules. The Romans—look at the buildings they built. How did they build those buildings? Using their heads and intuition. It's a mystery. Sometimes we check those old buildings, and everything checks. We check them with the most precise computations and they are just right.

Do you know how the Romans established the level? They would dig a ditch all around and pour it full of water—that gives them the level. There is such a thing as intuition, and apprehension. What does the word "apprehension" mean to you? Fear? But not necessarily fear?

Scott: Well, apprehension could mean being concerned. Or "apprehend" can also mean to perceive or be aware of something.

Pregnoff: That's correct. So you have to have intuition, and also have to have apprehension. You have to have a sense of—How am I doing? Am I doing it right? You have to watch yourself. You do it kind of unconsciously. Unconsciously you think—this looks kind of small, I'd better make it bigger.

Learning Engineering in a Good Office

Pregnoff: You get this by practicing engineering. Practice makes perfect. You have to practice your structural engineering. I don't think a college boy, no matter how bright he is, can open an office and begin to practice. He first has to work for someone else. That's what lawyers do. Young lawyers pay other lawyers. Pay them to help the younger ones learn the business. I think every young engineer should go to a good office and work with them for about five years, then open his own office.

Scott: Isn't that very important for maintenance of the standards of engineering?

Pregnoff: It is, but they don't do it the way doctors do. When a doctor graduates from college, he goes to a hospital and works as an intern. They work as interns for three or four years. In our practice they give an engineer credit for two years of college, as if it were equal to one year of practical experience. Thus, to get a license, two years of college count as one year of practice. But really it is not worth that much. Two years of college couldn't even compare with one year of practice.

Scott: You mean practice is more demanding?

Pregnoff: Yes, it's much different. In 1928 we were designing the San Francisco Opera House. The office engaged an engineer with a master's degree from the University of California. Our chief engineer, R.S. Chew, gave him the job of designing floor beams. Chew said to me, "Mike, give him the weights." Knowing the total weight on a beam, he can design the beam. The weights were given him, with the sketches for various architectural features. Then we left him alone.

About one and one-half hours later he comes to me and says, "Mr. Pregnoff, I don't even know how to start." I said, "I'll tell you," and I explained to him how to recognize various details on the plan and combine them into weights. College did not teach him how to read architectural plans. In college they give you beams and their weights and sizes, and you are asked to find moments and stresses. In practice you do not have sizes. You determine them by trials and computations.

Engineering Is More Than Computation

Pregnoff: You're asking me about my practice, and I will tell you my views on engineering. Engineering is not necessarily the computations. You don't build buildings by computations. No matter how good a mathematician you are, with that alone you're not going to design good buildings. You have to have structural experience.

Chapter 3

Seismic Design Considerations

"Some engineers do not detail the connections for adequate energy absorption. They do not provide enough ductility."

Brick and Steel

Pregnoff: When a building moves perpendicular to a brick wall surface, the brick wall without reinforcement will fall out. But when the force is parallel to the brick wall, the masonry gives and the steel frame takes the forces. If it was concrete, being very stiff, it would try to resist large forces and it would eventually crack.

Structural steel is a combination of beams and columns in bending, and is not as stiff as a concrete wall. But a brick wall is also not as stiff as concrete, so it gives a chance to the steel to carry the stresses. By the time the earthquake is over—maybe in 15 seconds—the brick wall will crack, but in general with lesser damage than concrete, and is easier to fix. That's why R.S. Chew believed in brick walls with steel frames.

Structural Steel Alone

Pregnoff: I believe in structural steel alone. I would not use concrete exterior walls. I would make all exterior walls of sheet metal. Walls of painted aluminum are flexible and they are also light. With metal wall panels, small bolts are sufficient. The panel is light and flexible; it only weighs 2 pounds per square foot instead of 50 pounds. Some buildings like that are built in San Francisco. One is on Market Street and one on Sacramento Street.

Now, however, they use precast concrete walls, which are heavy. They weigh 50 to 75 pounds per square foot of wall surface. They attach the walls with connectors, bolts—but these bolts may not move enough in the holes with small clearance, and the walls may crack and fall out.

Scott: So you especially like construction that uses steel, with metal wall panels?

Pregnoff: Yes, it's the best.

Redundancy in Seismic Design

Pregnoff: In 1946, an addition to the PG&E building was planned to be a structural steel frame with moment resisting connections. That was in 1946, and there was still no seismic code in San Francisco, but I designed the building for earthquake forces. I designed the building for the shear force at the bottom of the building equal to 5 percent of the weight of the building, plus 5 percent of live load per square foot. For the shear at the top I used one-third of the shear at the bottom.

Scott: Five percent lateral force resistance?

Pregnoff: Yes. It will give the trapezoidal variation of the shear along the height of the building. The trapezoidal variation takes care of higher modes of vibration during an earthquake. A tall building vibrates with a fundamental first mode and several higher modes—as many as the number of stories in the building. The architect, Arthur Brown, wanted concrete exterior walls with terra cotta facing attached to them. I said to myself that these concrete walls would be stiffer than the steel frames. Therefore, they would try to resist the earthquake forces first, whereas the steel would not resist the earthquake unless and until the concrete walls start to fail and crack, when the steel frame will begin to resist all forces. By "failing" I don't mean that the walls will collapse. They're reinforced, and they'll stick to the steel.

So I went to the chief engineer of PG&E and told him that I would like to design the walls to resist all the earthquake forces independently of the steel, and at the same time, I would also have the steel carry all the earthquake forces. The extra cost could be $1,500 to $2,000. He said, "Mike, this is the way to do it," and that is the way I've done it.

Scott: You were building redundancy into the design—having two systems independently able to resist seismic forces.

Pregnoff: That PG&E building was a 1946 addition to the existing 1925 building [designed by C.H. Snyder] on Market Street. Later they built another addition, which I did not design. I made a separation between Snyder's 1925 building and my 1946 building. The linoleum buckled at the separation during the Daly City earthquake of March 22, 1957, but there was no structural damage to either building. However, the Daly City earthquake didn't cause much damage in San Francisco.

Ductile Design Can Save Buildings

Pregnoff: The way the code is written now, it does not guarantee that a building will not be damaged in an earthquake. It [a building designed to code] may be damaged, but it will not collapse. We cannot use elastic analysis to provide for a very severe earthquake. The building would have to be built like a battleship. We contemplate that the building will be damaged, and in the very extreme vibration the steel will go beyond the elastic limit and go into yield. Yield would absorb the earthquake energy.

The rules are not strong enough to resist earthquakes in the elastic state of structure. Therefore, codes are written in the expectation that in a very large earthquake a building will begin to crack, maybe quite a bit, and the steel will go into the ductile range beyond the elastic limit. Codes contemplate that ductility will develop, and energy absorption will take place to prevent collapse.

Ductility saves the building during intense ground motion, when some members thus deform beyond the yield limit. That is what we call ductility. Our codes contemplate this ability of a structure to be useful beyond the yield limit. At critical peaks of ground motions the steel in some members will begin to enter slightly into a rather long plateau of constant "yield" stress, deflecting, but still stable, and not collapsing.

But the stress in some other members will not necessarily be at yield, due to strong influence of material variability, residual stresses, detailing, workmanship, and different local assistance from the nonstructural elements upon the steel bents. So what saves the building is this—its concrete will crack, but the steel will elongate and go into the inelastic range—that's ductility—and ductility will soften the energy, absorb it, and then the earthquake will be over. There has to be a very large sustained ground motion to bring the entire building into a large distortion and collapse it. When an engineer designs a joint, it is his responsibility to provide that ductility will be maintained. But some engineers do not detail the connections for adequate energy absorption. They do not provide enough ductility.

Scott: You are referring to designing a joint so it remains ductile despite stress?

Pregnoff: Yes, with proper lapping. For instance, in flat slab construction, in my jobs all bottom bars of slabs, at the columns, pass beyond the columns and lap. According to the code, you don't need to continue all bottom bars. Only 25 percent go through, and the rest of them stop. But our office would carry them all through. We want to tie the building together as much as possible to resist quake forces.

Also, under some conditions, we know the limitations of our knowledge, and the variations in quality of materials. Since you don't know what kind of earthquake will occur, we just designed conservatively. Our structural cost was perhaps as much as 5 to 10 percent more, but what does the 5 or 10 percent represent? Structural cost is only 25 percent of total cost of a project. So 10 percent of 25 percent is only 2.5 percent, and that is money well spent. I believe engineers should look upon the code as a minimum requirement, and in some cases go beyond this requirement. We did that.

Here is a picture from the Mexico City earthquake. [Points to photo of collapsed building in 1985 earthquake, which appeared in *Civil Engineering*, January 1986.] Look how that fell apart. Where's the tie there? If Degenkolb had designed that building, it wouldn't have happened that way—that's my opinion. You see such a small amount of reinforcing bars—meager.

When I see a picture like that, it appears to me that the building was not well tied together. Otherwise it maybe would distort, but is not going to collapse. They probably said, "Well, you don't need the reinforcement there, because it's in compression," so the reinforcement didn't even go through.

Scott: And there was the combination of the duration of that earthquake, the vibration went on for a minute or more, and the long-period motion.

Pregnoff: Yes, I know, I admit the earthquake's duration. But when you look at the photo, the concrete looks maybe poor, too. It appears to me that not enough ductility was provided in those concrete buildings. When things elongate they begin to absorb energy. When a column yields, it means that less energy is imparted to a building. Those details didn't provide ductility. Their code's pretty close to ours. They met code requirements, but did not meet the requirements of ductility.

Damping and Nonstructural Resistance

Pregnoff: Nowadays, they begin to use a concrete wall as a member which carries lateral forces. In those days they didn't use it. It was just considered as a partition. Also a lot of hollow-tile partitions were used. When an earthquake occurs they will try to resist the forces, but eventually will crack and fail. But those are nonstructural elements, and all buildings have them, and they help to save a building from catastrophic events.

Scott: You are talking about nonstructural elements in old buildings?

Pregnoff: Yes. Those old San Francisco buildings that you saw still standing in that 1906 postearthquake picture in the Freeman book, they're all here and still being used. There is a difference between modern multistory buildings now and multistory buildings in the past.

Multistory buildings in the past had nonstructural partitions. Some of them were of tile, some of plaster. Nowadays, buildings are built without partitions. The client comes in afterwards and puts the partitions in. They are of thin metal steel studs with gypsum boards attached to them. In the old days solid partitions reached from the floor to the floor above. Nowadays most of them do not go to the floor above.

When an earthquake shakes the building, everything is working—except you and me, we get scared—but everything is working and working to its utmost. Nonstructural partitions may begin to crack, but will carry some load. Even friction between cracks will carry some force, producing damping and absorbing energy. Steel alone will be like a perfect spring and may synchronize with the ground movements. Nonstructural partitions do not let the steel structures respond ideally during vibrations. The old buildings were saved by the participation of the nonstructural elements.

Scott: Henry Degenkolb emphasized to me that the effects of nonstructural or nonbearing elements can be very important design consideration in practical earthquake engineering.

Pregnoff: Yes. Those elements have to work. They do the best they can and they resist forces. Degenkolb felt that in every one of his buildings he would like to put some concrete walls, nonstructural, around stairwells, around the elevator. He likes to have those walls. They're not computed to resist lateral forces, but they give you extra damping for the building, so the building doesn't act ideally, like a spring. The worst thing you could design is something like an ideal machine, which, when it's synchronized, gets into resonance with earth motion.

[The nonstructural elements] are the reason why those old pre-1906 buildings are still being used now. They were damaged during the 1906 quake, and were also damaged by the fire. They were remodeled, and the buildings are still being used. Also they probably will not behave badly in future earthquakes. They may behave better than some of those recently built complicated, irregular buildings.

When we shake the computer model of the building, it responds to the earth vibrations like an ideal elastic spring—the deflections magnify and the elastic forces come out usually about four or more times greater than the design code forces. [Yet] observation shows that regular steel buildings don't collapse in an earthquake. Why? Well, the reason is—if you apply a force to a vibrating system, it will vibrate forever, until something stops it. Damping would stop it. If you take a spring and vibrate it, internal damping stops it. Buildings have internal damping. Also the nonstructural walls are going to try to take the forces and produce some damping.

So what do we do? We get forces [say] four times greater using dynamic elastic analysis. But we have ductility. What are you going to do with such large dynamic forces? Let's divide the forces by four. We call it the ductility factor. We divide by four, and get the forces [down] to about the same as the code forces.

Steel resists everything, like 450 Sutter, in San Francisco, which Snyder designed. It is a tall building, 30 stories. The outside walls are concrete, [and that] helps produce damping. And the interior partitions are going to work, too. Still, the steel carries everything. The bare steel carries the forces as though the concrete walls didn't exist.

Codes Provide Only Minimum Requirements

Pregnoff: I have seen the drawings of some engineers who just design their buildings so they comply with the code. They don't think things over. The codes cannot take care of all conditions. The codes give only the minimum requirements, and good engineers would add their own extra strength. They don't design just for the minimum. But instead of being conservative by using judgment, nowadays an engineer can become a businessman and build up an office of up to 200 men. He can employ computer experts having doctor's degrees, and who are mathematically proficient. They use the results of the computer with blind faith. They do not try to reevaluate the theoretical computer results to see if they look reasonable. They assume that the computer gives the right answer. The computer will give you the right answer, but only provided the input is right.

You have to visualize the action. When I analyze existing buildings, I consider which columns first begin to yield, while the others are not yielding. Then more force is put on and some more columns begin to yield, but the further group of columns may not yield, and then the earthquake is over and the building doesn't collapse. In the old days [1906] buildings were not irregular, they were simple.

Limiting Deflection and Drift to Reduce Damage

Pregnoff: Now, with all my experience, I believe—we both did, Bob Matheu and me—that building code requirements are only the minimum requirements. The code cannot take all conditions into account. Therefore when we consider a building moving during an earthquake, we do not want one floor to move with respect to the other to such an extent that it will crack all the partitions. Back in, say the years 1930-1935 or so, many engineers didn't even compute the deflection of the stories.

In short, when we design a building, we design so as to limit the deflection of one floor with respect to the other. Do you know what that means? That means that the stresses in the members are very much less than those allowed by the code when one designs for strength only. That means that the structure we design is sort of over-designed with respect to the building code, because deflection criteria governs the designs rather than the code strength criteria.

If I apply the lateral code forces to a building, I would arrive at a certain size of a column and certain size of beam. But if we build that way, then the building will deflect too much and ruin the interior work in the building—that costs more money than my structural cost. The structural cost is only 20 to 25 percent of total cost. It may cost up to 50 percent of the original cost to repair the building. In addition, you lose the tenants, who have to move out.

Scott: You are referring to having to go in afterwards and repair or retrofit a building after an earthquake, due to damage caused by excessive drift and deflection?

Pregnoff: Yes, repair after an earthquake. I say we believe that any buildings designed by us will deflect less than ones designed blindly by the code.

Scott: So the buildings you designed should survive an earthquake with much less damage?

Pregnoff: That is correct. In fact, the drift limitation has been given [by the code] only for the last 25 years. Before that, the drift limitation was not in the code at all. The first codes on earthquake design had no limitation as to drift. Now they give the limitation, but the limitation still is not small enough.

In 1930 the engineer H.V. Spurr, in his book *Wind Bracing*, called attention to proper rigidity of tall buildings.[4] He gave the criteria for deflections which will be tolerable to occupants. In a New York hotel [let's say that] a man on the upper floors begins to shave; then he looks at the bathtub and sees the water moving because the building is moving. He almost gets sick. He wants to move downstairs, or he moves out of the building. And that's just from the motion caused by the wind.

4. Spurr, H.V., *Wind Bracing: the Importance of Rigidity in High Towers.* New York: McGraw-Hill, 1930.

In 1981 we checked a hotel in San Francisco that had been designed and approved by City Hall. The bank wanted some experts to check it for earthquake. They engaged a firm to do it, and the firm engaged me to assist them as an expert. We analyzed that building using the aid of the computer and found the deflections were too large. The deflection was as much as 1 inch per story, and it's 35 floors, so it can move a total of 35 inches at the top. What really hurts the building is not just the total movement at the top, but the story deflections. Large deflections between stories will damage nonstructural elements. They corrected the problem in the hotel building by strengthening the beams and columns.

Designing Above the Code: Structural Costs Not Significant

Pregnoff: Engineers have to rely upon their past experiences and judgment, which dictate the necessity to be conservative. The engineers should remember that the code requirements are minimal. They have the right to increase their design—no one is going to sue them for that. In any case, most of the time when you do that, the structural cost of a building increases. The structural cost could be 20 percent higher. But this is only a small increase, because the structural cost is only 25 percent of the total cost of a project, so an engineer who designs for twice the code forces is not too extravagant.

A consulting engineer in Vancouver published an article in *Civil Engineering*, American Society of Civil Engineers, issue of January 1986, in which he writes, "My guess is that the codes are inadequate." Well, he says it now. But we [Pregnoff and Matheu] always thought of the code as providing only minimal requirements. To begin with, not everything in the code is right for every condition. So we always used the code in our own way. We did our own requirements, established how much some portions of the building should move with respect to other parts. Maybe the code says all it will move is 1 inch, but we say we don't want 1 inch of movement, we want only one-quarter inch of movement.

Here in the *Civil Engineering* article the author writes, "My guess is that a structure designed with dynamic analysis based on the expected levels of ground motion, and a comfortable safety factor for the ductility levels, would cost between 50 percent and 100 percent more than a structure designed under existing building codes. But, since the structure costs only about 20 percent of total building project cost, the additional cost would only be between 10 and 20 percent."

If you increase the structural costs of a building by 20 percent, it doesn't mean anything—nobody would know the difference. Another thing, too. When you make an increase of 20 percent, and you make the details all alike—repetition—the job gets cheaper. With the "economical" way, things become smaller, hard to connect, more complicated. If you make it bigger, it is easier to connect. It is the cost of the workmanship which counts—material weight doesn't count as much. Of course it increases the cost—steel costs so much—but workmanship costs three times as much as steel.

I am very much original that way. My partner too. I'm telling you, we, as engineers, take responsibility. Life is in your hands—I always feel that way. Therefore, I should be conservative. And I believe in conservatism. I believe that in some cases minimum requirements are not enough.

Chapter 4

Design Simplicity and Building Behavior

"Simplicity is important—a building should be simple in plan."

Building Makeup Determines Behavior

Pregnoff: The architect really decides the fate of the behavior of the building, not the engineers. We engineers have to adapt to situations and try to do the best we can.

The makeup of a building is the most important thing in its behavior during a quake. Simplicity is important—a building should be simple in plan. Any building that is irregular in plan cannot behave properly during an earthquake. L-shaped buildings are absolutely not good.

At the present time, however, I personally think that, instead of avoiding irregular buildings, some engineers design them because it suits the ideas of architects, and because the engineers have the assistance of the computer. They think that with the computer they can analyze anything. So they are relying on computer analysis to build the most complicated buildings, which will not behave properly.

I designed a lot of buildings in the old days. When we had an L-shaped building, we separated it at the juncture of the L—it was separated into two buildings, acting separately. In effect we then had two rectangular-shaped buildings. In 1925 [when there was no earthquake code in San Francisco] Chris Snyder designed the multistory PG&E Building on Market Street and Beale. Then, in 1946, PG&E wanted to make an addition to the building. The addition would have created an L-shaped building in plan, so I told the architect to separate the parts of the L by 6 inches.

I had a case recently when the client of an engineer friend of mine—who worked for me years ago—wanted to buy a recently built shopping center. The client wanted to buy it, but asked whether it was good for earthquakes. My engineer friend asked me to help him in the evaluation. The project consists of three one-story buildings, each about 550 feet long. In general, the long wall consists of very small steel tube columns with glass windows in between. The other long wall is a concrete block wall without openings. When earthquakes shake the building, the stiff block wall will resist practically all lateral forces and the building will torque. During an earthquake the earth movements will not be equal along the excessive length of the building, and there will be a tendency to damage.

The shopping center in general complies with the requirements of the code, which does not limit the length or the presence of torque. Most of the space is already leased and the stores are doing business. We have to be careful about condemning the building, which is legally safe, because it was approved by the city. We reported that the building was an irregular type of building. The engineer cannot make a highly irregular building behave well in an earthquake. Again, it is desirable to separate such a building into simple portions by using construction seismic joints. Also, to minimize the damage during earthquakes it is advisable to divide long buildings into shorter sections by using seismic joints.

Scott: Because of the separation, each component of an irregular building behaves as a separate, simple structure?

Pregnoff: Yes. In my practice, we separated them. We used sliding joints—the idea being to make the building simple in plan. In designing, if you see something like an L-shaped or irregular building, you have to separate it into regular shapes.

Simplicity and Repetition

Pregnoff: If a building is very simple, it acts simply. But if it is complicated, you don't know how it's going to act. So be conservative, make things alike. All members are alike. But now suppose that this member were a little longer, and that member still longer, on account of variations in columns? They'd have to manufacture them differently. Mine are all the same. It means repetition.

For example, in a concrete building, say I make all the beams 30 inches deep, but they would make adjacent beams on the shorter span 16 inches deep. That's a mistake. They should be 30 inches, all of them 30 inches. When the contractor builds the forms and shores, they should be all alike, to save the labor. In my case all steel bars are continuous because the beams are of the same depth. They produce a good continuous tie. This ties it together. The details are very important. The job should be detailed for the earthquake forces. [Then] when the contractor prepares the same detail for the whole floor it is a matter of repetition.

This uses more concrete, but it's cheaper in labor, and is a better job—has continuity.

There are two things in engineering: one thing is computation, and the other is creating detail. The detail may not necessarily comply with the requirements of computations. Besides strength, the details should also be considered. As far as the practical end is concerned, sometimes a shallower concrete beam should be made deeper, when the beam next to it is deep. Forms and shores should be repetitive. Thus, you save labor if the adjacent beams are of the same depth. The total labor cost in a building is four times greater than the total cost of material. It used to be two times, but now it's four times.

Scott: So you could say that labor costs are really the bulk of the building costs?

Pregnoff: Oh, sure, absolutely. Carpenters' labor costs are $40 an hour. That includes profit to the contractor.

Russian Seismic Code

Pregnoff: On its first page, the Russian seismic code states that all buildings should be regular, with a regular disposition of masses and stiffnesses. Of course, that is a country with one boss—and they want to make buildings as simple as possible, as economical as possible. They don't want to build a monument for themselves. Some architects here design very good-looking buildings, but they are not suitable for earthquake localities. What you build in New York, you should not build in San Francisco. Architects should adapt themselves to our more severe conditions on buildings.

Scott: The basic design of the building should take this area's seismicity into consideration?

Pregnoff: That's right. Very long buildings shouldn't be allowed. During the El Centro quake, as I recall, during 1/4 second, the earth moved 6 inches back and forth at the location of the instrument. I'm positive that during an earthquake, all points along the length of a building will not move the same 6 inches, and they may try to tear the building apart.

The longest building allowed in Russia is 70 meters. Seventy meters is about 230 feet. The long buildings in the shopping center I was talking about earlier were 550 feet. The design met code requirements, but the code doesn't take all conditions into account. The code gives you minimum requirements. And they probably analyzed for torque and everything. City halls don't have enough personnel to check our buildings. It takes six or eight months to design a building, and they have only a couple of weeks to check it. How could they check everything? They just have to trust the designer.

Schematic Simplification

Pregnoff: You have to visualize your structure. I imagine my building in my mind. In a book called *Structures*, Luigi Nervi, a famous Italian engineer who designed beautiful structures, talks about college education and what education should be like. He thinks education should go beyond the mathematics, and should be something else.[5]

Nervi is an engineer and architect at the same time. Let's see what Nervi says [reads from Nervi book]: "The formative stage of a design, during which its main characteristics are

5. Nervi, Luigi, *Structures*. F.W. Dodge, New York, 1956.

defined and its qualities and faults are determined once and for all, cannot make use of structural theory, and must resort to intuition and schematic simplification." So he says use schematic simplification when you plan a building. He built models too. Sometimes computation is not reliable, and so he built a model.

Nervi's design [process] is not much different from ours. In the process of design, when you start a building you conceive the detail first. You don't calculate the detail. Calculation does not give you size. You determine size by trial, and calculation checks your size choice. In the preliminary design stages, Bob Matheu and I established the sizes, using our intuition, our experience. Then we would give the job to our engineer to carry out the design.

A Practical Example: Corner Columns

Pregnoff: With a lot of things, you can intuitively simplify the behavior of the structure.

Scott: Could you give another practical example?

Pregnoff: Corner columns. During an earthquake, the corner column in a building is subjected to complicated biaxial bending due to two beams framing into it at right angles to each other. It also suffers badly due to overall torque of the building, which is induced by the rotational movements of the ground. Column shapes are not very effective for biaxial bending. Theoretically, the corner column would come out rather large in size. Do you know what we do? We make it small in dimension; just strong enough to carry vertical loads.

Scott: You make the corner column small?

Pregnoff: Small. When an earthquake occurs, the other columns will take the lateral load, and the one in the corner just rides, taking insignificant force, because it is small and very flexible. It adjusts itself without being overstressed, while the other columns are resisting the earthquake.

Scott: The columns that are not at the corners do the resisting?

Pregnoff: Yes. And yet some engineers will make computations and determine on a large, rather expensive column which may fail, due to inefficiency or inability of the shape to resist biaxial bending.

Scott: By choosing the larger corner column they made it more vulnerable because it gets more force than any of the others?

Pregnoff: Sure, [it is more vulnerable] because the designer made it large and stiff. You need to intuitively simplify structural behavior. It is nice, too, for the research people who may read our discussion to realize that besides the theoretical computation, there is such a thing as schematic simplification. You can make a job simple if the architect goes along with you. If he has trust, confidence that you are right, he may agree with you.

A Very Irregular Building

Pregnoff: There is one multistory building in San Francisco that is somewhat round in plan, and the architect wanted an atrium—a lot of light—so cut out a portion of it in the lower stories. The building is like a cylinder, with a portion of it cut off.

Scott: The cutoff cylinder shape makes a very irregular building?

Pregnoff: That's right. They built it that way. Computer analysis gave them an answer. Without the computer, it would be very difficult to analyze. Present code regulations—the Blue Book provisions—do not apply to the irregular building.

Scott: When you say the code forces do not apply, you basically mean that the designers have to be even more careful with buildings like that?

Pregnoff: Yes. When you make a dynamic analysis, for either a regular or irregular building, you use the ground movements, like of the El Centro quake. We have measurements of the El Centro quake—how it shook the earth. You make a mathematical model of the building, and using the computer, apply the ground movements to its bottom. The computer solves the problems and gives the response forces.

Irregular Buildings and the Current SEAOC Blue Book

Pregnoff: At present I'm working on the Seismology Committee of the Structural Engineers Association of Northern California (SEAONC). The Structural Engineers Association of California (SEAOC) has a Seismology Committee in all its sections: Northern California, Southern California, Central California, and San Diego. All four of these committees are combined into one committee of SEAOC, which reviews the work of the four associations.

The Seismology Committee is divided into subcommittees. We have 135 members, but only about 20 members attend. I attend every meeting, although maybe have missed one or two. We discuss all kinds of problems. Sometimes we do not agree. So we had a lot of north and south discussions. We have to compromise.

Scott: These compromises are made in the course of getting something finally adopted in the SEAOC Blue Book?[6]

Pregnoff: That's right. We [the SEAOC Seismology Committee in 1986] are now revising the present Blue Book, which is the Structural Engineers Association's rules to design for earthquakes. The new Blue Book will come out maybe in a couple of years. We've been working on it four years already. It's more severe, but not much. The Blue Book rules apply to regular buildings. The present Blue Book says that for buildings with irregular distribution of masses and stiffnesses, the forces given in the Blue Book do not apply.

But we allow such irregular buildings provided they're designed by dynamic analysis. Such irregular buildings should be designed with consideration for the response of the buildings to earth movements. Our committee has a notion that if a building is irregular, we should tell them [designers] how to overcome the problem. I am arguing that that's not right.

In my opinion, the code should not legalize the irregular buildings by giving the rules, without limitations on irregularity. Thus I believe that the code should not give the method, not tell the designer how to do it [build an irregular building]. We have practically finished the new

6. The "Blue Book" is the name by which SEAOC's *Lateral Force Requirements and Commentary* is generally known.

draft of the Blue Book, but I do not like it because, instead of discouraging irregular buildings, it gives solutions, which designers can put in the computer and then build a building. Also the new draft does not limit the irregularity. If the designer does not fully understand the problem, we should not give him the solutions. A not-knowledgeable man can apply those rules without knowing what he is doing. He should possess the knowledge, be able to think things over, and use his judgment and comprehension.

Scott: You mean that before even tackling an irregular building, the design engineer needs first to have already developed some real earthquake-engineering competence? That means being fully capable of thinking through the ways in which the building being designed will be affected by earth movements, and how the building and its various component parts will respond, hang together, and resist failure, despite shaking?

Pregnoff: That's right. As an engineer you have to remember that life is in your hands, so you have a responsibility. You cannot simply say, "I complied with the code." As I emphasized before, the code provides only minimum requirements.

Our proposed Blue Book edition says in effect: "In a simple building, apply the code forces and methods. However, if the building is irregular, use dynamic analysis." As if dynamic analysis solves the problem. Dynamic analysis may not solve the problems of a very complicated building. But our proposed Blue Book "legalizes" the highly irregular buildings.

Don't Build Complicated Buildings

Pregnoff: I say, if a building is too complicated, don't build it at all. They shouldn't allow irregular buildings, complicated buildings; they should make them regular. I've seen a picture of a Los Angeles building, a 25-story tower with a large two-story garage structure connected to its side. In an earthquake the garage structure will try to resist part of the motion of the tower, with resulting complicated torsion. If I were the designer I would have separated the two-story garage, and let the tall tower move independently on its own. You could put in doors and use a steel sliding plate in the floor at the doors.

Russian earthquake codes are different from ours. They give you a lot of details. But one is not allowed to design an irregular building there, unless one has a record of the behavior of a similar building in past earthquakes. They want symmetrical buildings—they don't want fancy buildings.

Scott: So the Russian requirement for simplicity is written into their code?

Pregnoff: It says something like: "The building shall have equal distribution of masses, symmetrical arrangements of resisting elements...insofar as possible." That's the goal.

The New Draft Just Says: "Use Dynamic Analysis"

Pregnoff: Our proposed new Blue Book does not suggest that buildings be made as simple as possible. It just says, "If you have a complicated building, use dynamic analysis," and also gives methods of doing the dynamic analysis. There are several ways of doing dynamic analysis, and there's a lot of uncertainty involved. In dynamic analysis you have to model the building. But the model cannot possibly really represent the building. It only represents a bunch of columns and beams having areas and other technical properties. But I also want to know how the members are connected and how they interact within a joint. A joint distorts. They try, by computer, to imitate the action of a joint. In the future, however, I think the computer probably will supersede human thinking. They won't be doing things my way—they'll be doing it by the computer.

Difficulties of Applying Dynamic Analysis

Pregnoff: It's very hard to judge irregular buildings, but we allow them provided they're designed by dynamic analysis. In fact, we have a Blue Book chapter on dynamic analysis. There's a dynamic analysis which analyzes a building beyond elastic limits. Also we tell them how to do dynamic analysis. Instead, I believe that the code should specify the forces, but not give the method, not tell the designer how to do it. A not-knowledgeable man can apply those rules without knowing what he is doing. I believe that the code should not give the solution to a problem. This is my opinion.

The rules of structural dynamics are very complicated. First, at the start, you don't know what kind of quake will occur. Second, you don't know the building, yet. When you create the building, you visualize its distortion according to some simple rules, rules in the code. In some cases you make it a little bit more than the code, but you make the thing simple.

If you educate the architect, explain it to him, if he understands why it is so, you get good teamwork and produce a good building. Now in the proposed Blue Book, we won't discourage complicated, irregular buildings. We state that if you have an irregular building, you use dynamic analysis, as though the dynamic analysis will represent the actual behavior of the highly irregular building.

Independent Review

Pregnoff: I just read in the civil engineering magazine, published by the American Society of Civil Engineers, that some firms, at the start of the design of a building, ask another firm or senior engineers [older engineers], to see if they are on the right track. The tendency now is to have a second opinion. I see nothing wrong about that. I thought the new version of the Blue Book was going to have a provision that with a very irregular building the designers should have a group of other engineers to look it over. But I think that provision was killed.

Evaluate Analysis Using Common Sense

Pregnoff: [But] as I say, the computer alone shouldn't give you the answer. You should reevaluate it and see if it is reasonable. For instance, if you're not so sure about soil characteristics, you give the computer three sets of characteristics. It will give you the results for all three. The engineers will then use their own judgment as to which set of characteristics to select. As Luigi Nervi[7] said, any mathematical solution should not be trusted without intuitive reevaluation. The solution may be right but the input was wrong. I would use the computer all

7. Nervi, Luigi, *Structures*, F.W. Dodge, New York, 1956.

the time, but I would have somebody to review the computer solution when it comes in. Check it out by common sense, that's all.

Any mathematical analysis, whether done by computer or by hand, should be reevaluated from a practical standpoint. To some computer men, however, a building is a bunch of lines that resist forces. One line has certain values of stiffness, and strength, another one has different values. And they put them into the computer and the computer program determines how the forces are distributed and gives an answer, gives them magnitudes of forces, etc.

Those results should be evaluated. You need to apply judgment in order to evaluate the answer provided by the computations, and to decide if the forces indicated are right. That's what Luigi Nervi said, the Italian engineer whom I mentioned before. In his book, *Structures*, Nervi said that every mathematical solution should be reevaluated from your own standpoint to see if it is reasonable. If an answer doesn't look reasonable, something may be wrong; do something else, to see if there is an error somewhere.

Thus, in my own practice, our computer men might do an analysis and give me the answers to review. Maybe I would look at the answers and notice, say, that the shear forces get larger towards the top of the building instead of getting less—so somewhere there is a mistake. I would start to study it to find the error. You have to look over everything.

Here is an example from about 1976. One of my engineer friends had gotten a $200 million job involving retrofit of a four-story building. They chose structural X-bracing—diagonal bracing—to resist earthquakes. They gave our office the job of making detail drawings, and they gave us the forces of computer analysis.

There are two diagonals, and when lateral force is applied, half of the force is carried by one member of the X by compression and half by the other X member in tension. The forces on each member should be alike, except that one is compression and the other one is tension. Instead, however, the forces given by the computer differed appreciably, so I called the computer man. He told me that the difference in forces was due to different modes of vibration. But I told him that instantaneously at specific times in each mode, the forces should be alike—should be exactly equal—except in reverse. Later the computer man called me up to say they had made a mistake. So they corrected it.

Here is another example involving the multistory Kaiser Hospital Building on Geary Street, in San Francisco. We were making a report on its ability to resist earthquake forces. It has a thin concrete wall, only 6 inches thick, and about 50 feet long, just a yard wall, attached to the building. Our computer expert made an analysis and said the building was overstressed in the bottom story, because the wall was overstressed. My partner said, "This is just a yard wall. You could separate it and throw it out of the analysis. What do you put it for?" He took it out of the analysis, and the building was okay.

Scott: But in the original computer analysis the yard wall had been treated as if it were an integral part of the basic structure?

Pregnoff: That's right—he treated it as though it were a long, stiff structural element. Quite often a building will have a wall situated in such a manner that it is overstressed. Then you know what you do? You go ahead and build it, but you separate it, so the forces won't go into the wall. Make a slight sliding joint. Or you say, let it crack. It will crack, but nothing will fall down because the other elements are resisting the forces.

Pregnoff Memo: *The Engineer and the Computer Age*

Pregnoff: Now I will read some of my thoughts to you [reading from his memorandum[8]]. In the memo I'm asking:

> Can the buildings be designed by application of the mathematical formulas of structural mechanics? Or instead, should they be designed by intuitive evaluation of the theoretical results, taking into account the wide variation between the theoretical assumptions and the actual properties of materials—concrete, steel, wood, soil?
>
> Structural engineers should know that the computer model of the structure is only an approximate picture of its behavior. The engineer should rely upon his intuitive knowledge, apprehension, and experiences to visualize the actual behavior of the structure.
>
> The structural engineer should be capable of evaluating the limitations of the computer output, based on the mathematic model of the structure. In fact, he should be intimately connected with the conception of the model. The structure includes the layers of the soil upon which the structure rests.
>
> To the uninitiated young engineer, the more complex the theoretical model, the more it represents the truth. This fact gives him blind faith in the results and relieves him from responsibility of thinking things over.
>
> Today the young engineer may be rated best if he knows how to set up computer programs. He may also be rated high if he has a Masters or Ph.D. degree.

Scott: That expresses your philosophy on computer use in design?

Pregnoff: Yes. That is my philosophy.

Scott: When you wrote this memo in 1975, was it done mainly to put your basic philosophy in writing?

Pregnoff: I did it for the following reason: The Structural Engineers Association of California has a meeting every year, and there was to be a meeting on "The Future and the Present of Engineering." I gave this to the program committee to be used as one of the topics.

Scott: How did they respond? I presume this was all done in preparation for the annual meeting around 1975 or 1976?

8. Unpublished memorandum by Michael V. Pregnoff, "The Engineer in the Computer Age," November 13, 1975.

Pregnoff: Well, I gave it to the [program] chairman, who worked for a big organization. He said, "Oh, Mike, I have a lot of trouble with this computer business." So he wasn't so enthusiastic about putting my topic on the program.

The Future: Computer Used Like Handbook

Scott: What do you think of the future use of computers for irregular buildings, as more capable computers are developed, along with the far more sophisticated kinds of analysis that will be possible when such computers are more plentiful? Especially, do you think they may then be able to handle the problems of very irregular buildings more effectively?

Pregnoff: Probably. But the practical aspects will still be controlled by the engineers. In general I think that in the future they will use the computer like a handbook, like a cookbook. In FORTRAN language they have a sheet with 80 columns. You enter into it the dimensions of a building, number of frames, stories and you also input the assumed earth motions back and forth and vertically. Then the computer will solve the problem and even give drawings in full size of several styles of joints. The computer will give everything. But it is a human being who will make the decision, for example, that [in the interest of uniformity] a short beam in a concrete building should be made the same size as the adjacent long beam.

You ask the computer to give several solutions for a given condition. That is, a few beam sizes with different reinforcement, etc. You pick the one you want. The program is such that you ask for ten beams, ten sizes with different reinforcement for the same condition. You pick out the one you want. Of course it costs money to analyze ten different beams, so you ask for maybe two or three beams. You ask for the desirable depth, it will give you that depth. Then you ask for another one that is 2 inches shallower, it will give you that with all the reinforcement.

Scott: So even then the engineer will still be in charge of the practical end of design?

Pregnoff: That's right.

Chapter 5

Seismic Code Development

"The code gives you forces and some details. When designers follow it blindly, it is okay for an average building—better than no code."

Early Days

Scott: I'd like your comments on the development of what is sometimes referred to as the "California practice." I'd also like you to talk about the *Separate 66* philosophy and the development of the Blue Book that came along a little after that; the early attempts to establish some standards of practice with special respect to earthquake resistance.

Pregnoff: In 1930 the structural engineers organized an association consisting mostly of engineers in private practice. Among the really active ones were [Henry] Brunnier, [L.H.] Nishkian, [E.L.] Cope, [J.B.] Leonard. Chew was not in here, because he was a loner. But Gus Saph, was, yes. Those fellows formed the association.

Back in 1923 and afterward, the engineers had begun to think about earthquakes. I remember in Snyder's office, Hall—who was my boss at the time—and Snyder talked a little bit about earthquakes. But the men like Chew, they practiced it. While we were designing the Opera House for Snyder—R.S. Chew was running it with me, so we put something into it for earthquake forces. So there were a few of us, a few engineers who designed for earthquakes—Saph probably, [Austin] Earl, and Cope.

1939 Chamber of Commerce Code: Designing for Lateral Forces

Pregnoff: Back in 1930 in California, partly as a response to the Santa Barbara earthquake of 1925, various committees of more than 100 technical men worked for several years and produced the *Building Code for California*,[9] nearly 500 pages in length. The State Chamber of Commerce appropriated money somehow, from somewhere, and in 1930 a committee of engineers was formed. Among them were Snyder and Nishkian. I know, because I was helping Snyder. It's a code that covers flat slab construction, steel construction, concrete construction—everything. It was published in 1939 by the California State Chamber of Commerce. They produced a very good code. I have a copy of it.

Scott: Is that 1939 code still readily available, or is it a collector's item?

Pregnoff: It's not available at all, and anyway it was never adopted as a code. It didn't go into effect.

The 1939 code proposed a peculiar way to design buildings for lateral forces due to earthquakes. What they did is this. Say you are designing a tall building. At the top two levels—the roof and the next level down—you use a lateral force of 8 percent of dead load (DL) plus live load (LL). At the next levels—the third and fourth levels down—use 6 percent of DL plus LL; at the fifth and sixth levels, use 4 percent of DL plus LL. At all levels below the sixth one down—counting from and including the roof—use 2 percent of DL plus LL.

The lateral force resistance at each level would be equal to a percent of the dead load plus live load adjacent to those levels. Suppose you have three floors and a roof [four supported levels], then at each level—counting from top of building [the roof counted as the first level]—the lateral force as a percentage of DL plus LL, is 8 percent, 8 percent, 6 percent, and 6 percent. If you have nine levels and a roof you use the following percentages: 8, 8, 6, 6, 4, 4, and then 2 percent of DL plus LL lateral force below the sixth level, counting from the top. Now in 1987 we are doing similarly, except with larger percentages of dead load only. So their way to resist earthquake forces was not bad. There is a dynamic effect, and this is a very good way to compute it. [See Appendix, *Excerpts: Building Code for California*, 1939.]

Scott: Although you called that old 1939 code's method kind of peculiar, you also are saying that at least this aspect of it had considerable merit?

Pregnoff: Yes. If you have a 2-story building, you use a lateral force [top two levels]. They put the quake force design in the appendix in that of 8 percent of DL plus LL at the roof and second floor code. Any community had a choice of designing for quake if they wished to do so at that time. That code was published in 1939, but as I said, it was never adopted [by any jurisdiction].

9. California State Chamber of Commerce, *Building Code for California*, ed. Edwin Bergstrom, 1939.

Scott: What effect did the Chamber of Commerce code have? Did it have influence, even though it was not formally adopted?

Field Act and Following

Pregnoff: I'll tell you what influence it had. After the 1933 Long Beach earthquake occurred, the state was empowered to check public school buildings. After that, you couldn't build a public school without approval by the State Division of Architecture. The Division of Architecture put out a little book, called Appendix A. Engineers were given Appendix A [Pregnoff pulls out a copy].

Scott: [Reading.] Regulation No. 5, "Relating to the Safety of Design and Construction of Public School Buildings in California." This copy says revised 1937, and the first edition of Appendix A was probably done shortly after 1933.

Pregnoff: Yes, maybe the original was in 1933 or shortly after. When the 1933 Long Beach earthquake occurred, it took them a while to organize. I don't know when it first came out. I understand that some rules out of those Chamber of Commerce committee studies [for the code published in 1939] were put into Appendix A. It is a pretty good little document. Appendix A was revised several times. It is now called Title 24. Those were pretty good little rules, very simple, not like the Uniform Building Code, which is now a little too complicated.

Scott: So the content of the first version of Appendix A for the Field Act was the principal effect of that code-drafting effort sponsored by the Chamber of Commerce?

Pregnoff: Yes.

Separate 66

Pregnoff: A short time ago, you asked about "California practice."

Scott: Yes, Henry Degenkolb emphasized to me the importance of California practice in earthquake engineering.

Pregnoff: When Degenkolb talks about California practice, he means that California engineers were more conscious of the quake forces. In 1951 the Joint Committee of the ASCE and the Structural Engineers Association of Northern California published a paper in the Proceedings of ASCE, "Lateral Forces of Earthquake and Wind," known as *Separate 66*. Then in 1959 the Structural Engineers Association of California published "Recommended Lateral Force Requirements," known as the Blue Book. The Blue Book was based on the principles of *Separate 66*.

Critics of Separate 66

Pregnoff: Anyway, the Structural Engineers Association wrote what is called *Separate 66*, published by the American Society of Civil Engineers. Some professors—southern California professors like R.R. Martel criticized it. George Housner criticized it and wrote quite a discussion. Also some Japanese criticized it. The critics thought maybe it was oversimplified, or something like that. But nobody paid much attention to them.

The critics said a lot of things were wrong in it. But in my opinion it was quite an advancement. *Separate 66* analyzes a single degree of freedom element.

Scott: Movement in one direction?

Pregnoff: Yes, a single degree of freedom. A structure responds to a quake in a certain way. For the slow movement it responds slowly. For fast movement it responds sharply. The stiff building with a small natural period, like 0. 2 of a second, will respond with a lateral force of 9 or 10 percent of its weight.

The flexible building with a long period of say 2 seconds will respond with a lateral force of only 4 percent of its weight. So in *Separate 66* the force is a function of the period, while the old codes had it as a function of the number of stories. That was a key difference. They gave the formula for the lateral force as a function of the natural period of a building. Frank Ulrich of the U.S. Coast and Geodetic Survey measured the periods of a lot of buildings in San Francisco and Los Angeles. They plotted a bunch of dots for periods of buildings with various ratios of height to width. From the average curve they obtained the formula for the lateral force as a function of the natural [fundamental] period of a building. That's in *Separate 66*. *Separate 66* is the first approach that is more or less advanced. That's where we started it, in California.

Blue Book and the UBC

Scott: First *Separate 66*, and then the Blue Book.

Pregnoff: Yes. The Blue Book is more advanced. I think it is really better than any other code. I don't know what is more logical. I don't know what Japan has.

Scott: To what extent has the Uniform Building Code adopted what is in the Blue Book?

Pregnoff: In the past they've adopted it fully. They just copied it. But now lately, a new version of the Blue Book is being worked on. I am a member of the committee. I don't know, maybe in a couple years it will be put out. It has more details in it, and is more advanced. South, north and central and San Diego engineers are working together. We argued a lot. It will be more conservative and more detailed, particularly on steel.

Scott: Will it be basically a better code?

Pregnoff: I don't think so. You know, the earthquake is so uncertain.

It Depends on the Engineer

Scott: Do you mean the code is too conservative?

Pregnoff: I don't know what's better. I say that with a building designed now, using the code, it depends on the engineer who designs it, not on the code. I say buildings designed by Brunnier or Degenkolb are better than buildings designed by some other engineers. No question about it. Because it isn't just that you follow the code, it is the *details* that you provide. The code doesn't give you all the details.

Engineering Still an Art

Scott: So even with improved codes and more advanced codes over the years, the code is still a "cookbook" approach, I guess.

Pregnoff: The code gives you forces and some details. When designers follow it blindly, it is okay for an average building—better than no code. But for a large, complicated building, I would say Degenkolb's building would be better than one by some other engineers, who are not experienced engineers. Yet they use the same code.

Scott: So the result still depends very much on the engineer?

Pregnoff: Always. That is the key factor. If you read the commentary in our Blue Book, it says that a lot of things depend on the engineer. And in books written by very fine professors, they always mention that this is still kind of an art. Newmark and Rosenblueth[10] wrote a book on earthquake design, and Ray Clough and Joe Penzien[11] wrote a book too. In their introduction Newmark and Rosenblueth say, "We face uncertainty because it is our task to design a structure about whose properties we know little, to resist future earthquakes, about whose characteristics we know even less."

John Blume

Scott: I would like to ask about your view of John Blume and his contributions to earthquake engineering. I am asking particularly since you yourself are known among the practicing engineers as being especially good at the use of math in engineering. I also know that early on John Blume probably did more mathematically-oriented work and computer analysis than just about any other practicing engineer in California.

Pregnoff: Blume and I are different. I'm not demonstrative. Blume is a real, what would you call it? I say he is a real star. In 1961 he wrote a book together with Newmark, and Corning—*Design of Multistory Reinforced Concrete Buildings for Earthquake Motions*—a beautiful book.[12] He wrote a paper entitled, "Structural Dynamics in Earthquake Resistant Design," in the ASCE *Journal*.[13] It's a masterpiece, and he got an award [the Moisseiff Award] for that.

I didn't get any awards. The only award I got was from Vice-Admiral Moreell, U. S. Navy, a meritorious civilian service award for doing a lot of Navy construction work during the war. I designed a lot of Navy buildings for the 12th Naval district, and they were very well satisfied.

I worked on various technical committees, and also, when I was President of the Structural Engineers Association of Northern California, I had to conduct monthly meetings. They liked the way I talked—often I would talk on human topics, and I was humorous. But I do not like to advertise myself.

Our office never solicited; jobs came to us from big architects, from mouth to mouth. I developed my method of analysis of tall frames, but I didn't publish it. People in Australia have it, and some friends of mine who worked for me are using it. A lot of people are using it, but I did not want to publish because it's approximate, maybe within 20 percent of real earthquake forces. You have to know how to use it.

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10. Newmark, Nathan M. and and Emilio Rosenblueth, *Fundamentals of Earthquake Engineering.* Englewood Cliffs, N. J., Prentice-Hall, 1971.
11. Clough, Ray W. and Joseph Penzien, *Dynamics of Structures*, New York, McGraw-Hill, 1st ed., 1975, 2nd ed., 1993.
12. Blume, John A., Nathan Newmark, and Leo H. Corning, *Design of Multistory Reinforced Concrete Buildings for Earthquake Motions.* Portland Cement Association, 1961.
13. Blume, John A., "Structural Dynamics in Earthquake-Resistant Design," in *Journal of the Structural Division.* Proceeding of the American Society of Civil Engineers, ASCE, New York, NY, July 1958.

You play conservatively. But anyway the building [designed that way] won't fall down.

Jacobsen and the Building Model

Pregnoff: Blume was a young Stanford University student at the time when he got acquainted with Professor [Lydik] Jacobsen and began to work closely with him. I think Jacobsen engendered in Blume ideas about earthquake design. Blume and Jacobsen built a shaking table machine. Jacobsen made a model of a tall building that C.H. Snyder designed, a mathematical model, and they shook it, and took movies.

Scott: If they took movies, then it must have been an actual physical model and not just a mathematical model.

Pregnoff: It was a mathematical model, but was not only on paper. It had model members, metal springs, which did not even look like building members. The model acted almost like a perfect machine because there was no damping, no plaster, and nothing helping to resist lateral forces, only the a bare model. That experimental model represents the mathematical model on paper; the springs represent the computed properties of columns and beams. They shook the model and measured its distortions, which represented the behavior of the model.[14]

14. Blume, John A., and Harry L. Hesselmeyer, "The Reconciliation of the Computed and Observed Periods of Vibration of a Fifteen-Story Building," Engineer's Degree thesis. Stanford University, CA, 1934.

Wind Experiments; The Alexander Building Analysis

Pregnoff: I remember when Blume was a younger man, at one time they thought the Golden Gate Bridge was oscillating up and down too much during winds, similar to what happened to the bridge in Tacoma. So John made experiments about bridge shaking. He hung a bunch of metal buckets, and with the buckets in the water tried to control the movement. He described those studies to me. So back then he was already trying, and he had an analytical mind.

Also Blume published a special analysis of the Alexander Building, in San Francisco on Sutter and Montgomery streets. It's isolated from other buildings, so he made a complete analysis of it, wrote a theory on how nonstructural elements are participating in it—a wonderful piece of work. He did that maybe in the 1950s. He was that kind. You cannot compare me with him. As I said, he's a star—from standpoint of theoretical engineering. He was a consultant in atomic energy planning.

I don't know whether he'll give you interview or not, but maybe he will. He likes to do things for people, and your work is for people, to propagate knowledge. Oh, he is tops.

Scott: You're talking basically about his own contribution to the field, especially the literature?

Pregnoff: His contribution to earthquake engineering. He's an earthquake man, and he was on many committees. Similar to Degenkolb. He's a very modest, very nice man.

He also sold his business. Maybe he was taken over, anyway the firm became URS. They

[Blume, URS] have done a lot of government jobs. [A few years ago] the state of California was going to reconstruct the Capitol in Sacramento—they wanted to strengthen it for quakes. I got a letter from the State Architect saying they'd like me to submit my experience, if I wanted to do that job. They came to my office and interviewed me. Also they interviewed John Blume's office. I had no chance with respect to Blume—and Blume got the job, which was quite a job.

Chapter 6

Observations on Prevailing Practice

"Simplicity of detail counts, not the amount of material you put into it."

Most Buildings Should Perform Well

Scott: How do you feel about the prevailing engineering practice? Has it advanced over the years, especially in seismic design? Or is it still only a relatively few who practice seismic design?

Pregnoff: I sort of disagree with Degenkolb, to the extent that he says many modern buildings may not behave very well, maybe thousands of people will be killed.

Scott: He doesn't say that about all of them, but he says some buildings are not going to behave very well.

Pregnoff: I think there will be fewer people killed, because in American building practice, I never see thousands of people killed. I don't see that. It's only seen in Mexico. We didn't see that in Long Beach, we didn't see that in Alaska. We haven't seen it in the U.S. anywhere, so far. Even in San Francisco, in 1906, I think only 400 or so have been killed. So really not so many people have been killed by quakes in the U.S. But maybe in the future, with tall buildings, maybe they will be. Also I somehow don't think the steel buildings will collapse, even if poorly designed.

On the Other Hand, Some Are Bad

Pregnoff: James Stratta, structural engineer, designed several buildings for a corporation. But one building they wanted in a rush. One contractor had the land already, near their plant. He said "I'll build the building using my architect and my engineer." So naturally Stratta didn't design it. Then when the contractor had practically completed the structural features, the corporation asked the insurance company for the rates. After their inspection the insurance people said, "We're not going to give you rates." "Why?" "The building is going to collapse during a quake."

The corporation got in touch with Jim Stratta, asking, "What happened? Why don't you look over that building for us?" Jim said, "I want some experts." So they got me and Degenkolb. We looked at the building. It met the code requirements. It had shear walls, but forces were not delivered to the shear walls. The beam sat on the brackets. Instead of using ties, there were no ties. The wall was considered as taking force, but there was no connection to deliver the forces. So we said that the building was not good. Each one of us made independent reports.

We had a meeting with the corporation board of directors. Degenkolb talked, Jim Stratta talked, and then I talked. Degenkolb said that there were no ties, but then there's no requirement in the code for the ties. He said that he was not sure if they would win the case, if they sued. Degenkolb told them that he was not sure that they would win, because the engineer had designed according to the code. My own talk was short—I said, "I've heard of an Englishman who said, 'I am not rich enough to buy cheap shoes.' You bought cheap shoes."

Details Were Inadequate

Scott: It complied with the code, but the design still was inadequate nevertheless.

Pregnoff: The details were inadequate. The engineer's computation was correct, but the details were not correct. The code doesn't tell you how to connect things. So they said, "Let's meet with the engineer who designed it." So we met the engineer. His brother was the architect. Fine drawings, but no details. They gave him all our three reports so he could get prepared. They asked him "What do you think?" He said, "Well, all these honorable experienced engineers, they're right, I have nothing to say. I complied with the code." So I asked him, "Did you scheme the job, did you give your scheme?" The engineer said, "No." He said he had wanted to make a monolithic pour, but the contractor wanted precast members—that was the contractor's scheme. "I had to follow his scheme, I used his scheme."

Scott: So the contractor was calling the shots on that, not the engineer?

Pregnoff: But the contractor still didn't violate the code. The design was according to the code, everything was designed to code. But still what the contractor got was disconnected pieces. Maybe somehow those pieces could act together—God knows, pushing against each other. But it's not good. So the corporation got Jim Stratta to fix that building. He put braces on the outside walls.

Poor Drawings

Pregnoff: They never learn. This happens all the time. About two years ago, a friend of mine brought to my office a print of drawings of a multimillion dollar structure he was inspecting during construction. The details were poor. So poor that neither I nor Al Paquette the engineer, could interpret them. The contractor had great difficulty in building the job. In the past I blamed the architects for earthquake problems. But lately, seeing some poor and incomplete drawings, particularly issued by large 100-to-200-men firms, I have begun to change my mind.

Scott: You don't think they're conscious of doing a poor job?

Pregnoff: I think the principals don't realize what they're putting out.

Large Offices: Quality Control Problems

Scott: There ought to be some quality control somewhere.

Pregnoff: You imagine having 400 or even 200 men. One would have difficulties to find responsible supervising personnel. In a large firm you need at least five or six leaders in order to have a good quality control. Yet the government and big institutions give the jobs to larger firms. Maybe it is debatable, but my opinion is based on observations. The firms with four or six employees do not get many jobs. Big institutions, government institutions, they give jobs to big firms.

Small Offices and Quality Engineering

Scott: It sounds to me as if some of the real quality engineering is done in relatively small offices.

Pregnoff: Because they put what we call heart and soul into it. They're interested themselves. They are putting themselves into it. They have responsibility. The owner of a small office in reality is a poor businessman, but he loves engineering and he is an above-average engineer. That's why he opened his office. He works himself, puts what we call his heart and soul into a job. He also works intimately with his employees. He takes interest and knows every job. He is aware of his responsibility. He tries to get a *name* for himself by doing a good job.

Maintaining Standards: Checking Jobs

Scott: So how do we maintain the standards of engineering practice while things are going in the direction of the very, very large offices?

Pregnoff: I would check their jobs. Every job should be checked thoroughly by the city, the same way school jobs are being checked by the state.

Scott: They should be checked as thoroughly as the checking done under the Field Act?

Pregnoff: Yes, but it is not done now. So maybe cities should be forced to have competent checking. If they haven't got their own checking ability, let them engage private engineers to check for them. That's the way to do it.

Scott: For that, I guess they would need reasonably well-qualified engineers, something more than just plan checkers who are only responsible for saying whether a design complies with the code.

As I understand it, you have checked quite a few buildings yourself. You've done quite a few schools—you did Oakland schools, though that may have been some time back. What is it like, dealing with schools?

Pregnoff: I had no trouble because I was doing it properly.

Simplicity and Repetition

Pregnoff: Simplicity of detail counts, not the amount of material you put into it. Repetition counts. C.H. Snyder, the engineer for whom I was working, had a big job for Washington D.C., the Interstate Commerce Building. Three engineers designed steel beams for different floors. C.H. Snyder said to me "Mike, we have a lot of beams of the same size and different weights. They are using 12"/28# and 12"/32#. Instead of two sizes, use one size, 14"/30#. Similarly with other sizes." I went over all the plans, and we came out with fewer varieties of beams. We saved money, because many beams were alike. That just shows you that economy is not necessarily economy in weight. It's in repetition. The contractor gets enthusiastic, if you simplify the sizes. Makes it simple to buy, simple to order, simple to detail. Sometimes you can save as much as 5 percent to 10 percent on a job.

I'm pointing out that engineering is not just complying with the code, not [just] complying with computations. There is something else—making them simple, repetitive, fast to build, is also important. You can design small beams that are difficult to connect, and then the labor costs more money. But some engineers, as for example some employed by big offices, may design by computer, and the computer gives them the smallest size as being economical, because of less weight.

"...Nobody Thinks Things Over"

Pregnoff: Some firms don't pay enough attention to the quality of their employees. In the old days, when the computer didn't exist, they hired somebody who had to be very experienced. He had to be reliable in every respect. Now, the low-paid man can punch the computer program. He uses a cookbook, which is written so that one mechanically enters the numbers without thinking. It is easier to get a job done, but the job may not come out good from the practical standpoint in the field. The design may lack repetition, and simplicity in erection, etc. Back then, the designer had to design. Nowadays, he uses the computer, punches in the program. In other words, for them it's easier to put out a job now, but the job is not as good. It is not as economical.

Scott: Nobody sits down and thinks through the basic design. Is that principally what you're saying is wrong?

Pregnoff: Nobody thinks things over. Sees if everything looks reasonable. For example, slabs on ground—they make them 4 inches thick. I never make them less than 5 inches. A slab on ground cracks all the time. What is an extra inch?

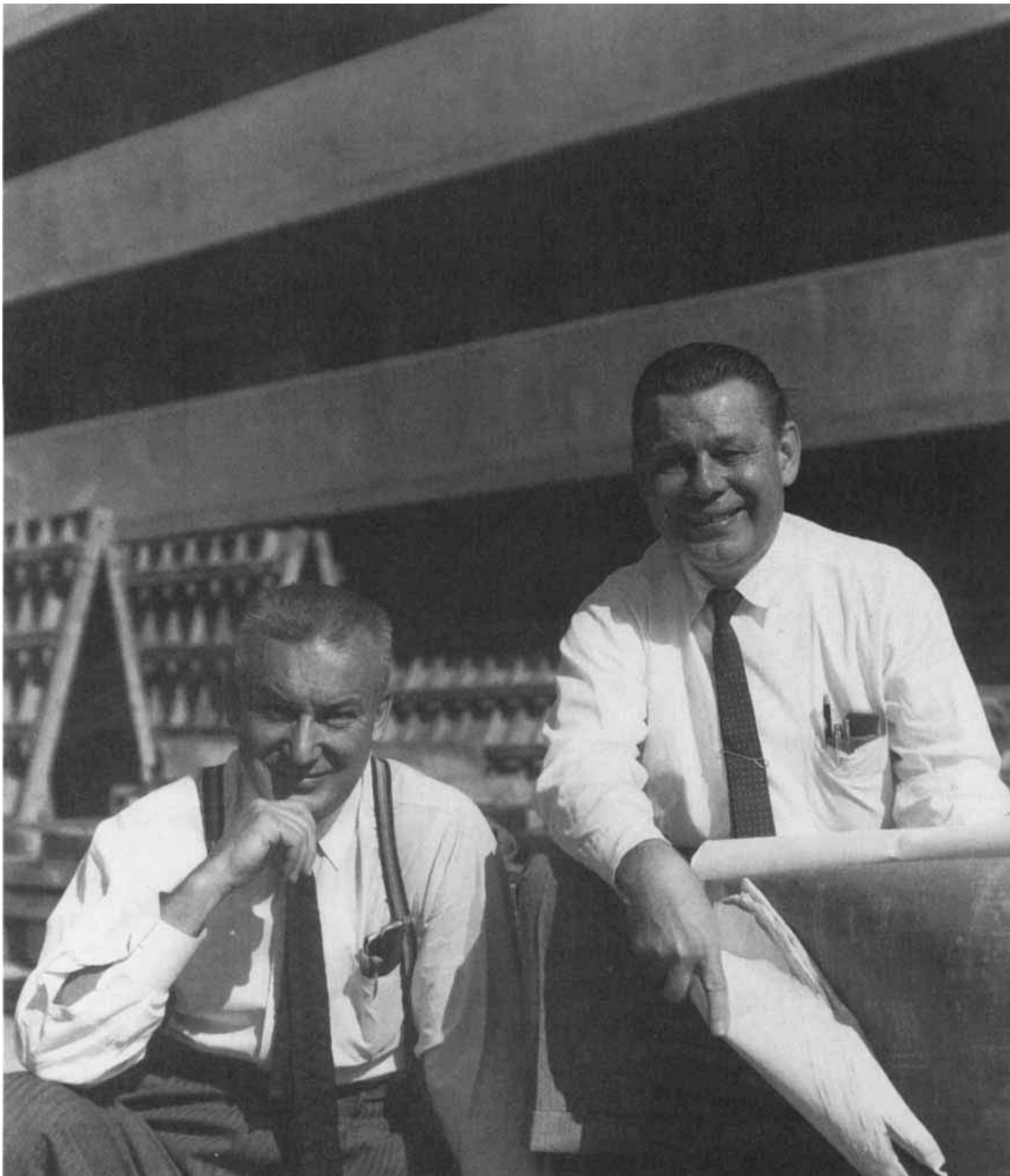
Scott: But that provides a better margin of safety. I take you to mean it can make all the difference in performance.

Pregnoff: Using 5 inches instead of 4 inches adds 25 percent, makes it one-fourth greater in thickness. Also it is hard to control the thickness with the erratic ground surface. With a 4-inch slab, some of it will come out 3-1/2 inches. But with my 5-inch slabs, sometimes it comes out 4-1/2 inches and sometimes 5-1/2 inches.

Scott: Well, here we are at the end of an all-day interview. We have covered a lot of territory in this long recording session. As I said before, it is quite unusual for an interviewer to schedule an oral history session of this duration, mostly because interviewees typically run out of steam after an hour or two. But your energy supply obviously operates on a different timetable. You're also a man of your word—you said you could outlast me, and you did.

Photographs

Michael V. Pregnoff, 1953 (photo: Moulin Studios)



Michael V. Pregnoff (right) and partner Robert Matheu, 1960 (photo: Russell Leake)

Appendix

Seismic Design Excerpts from the California State Chamber of Commerce Building Code for California, 1939

Building Code for California (1939)

California State Chamber of Commerce, Building Code for California, ed. Edwin Bergstrom, 1939.

Work on this code was begun in response to the 1925 Santa Barbara earthquake. The work was done by committees numbering over one hundred members, representing state and local associations of architects, civil engineers and contractors. The intent was to develop a "Uniform Building Code—California Edition," and to publish the result by 1930. The effort went more slowly than anticipated, however, and the first actual use of the results came when the Field Act, which enacted minimum seismic standards for California public schools, was passed in 1933. The regulations developed to implement the Field Act relied heavily on the seismic design work that had been done for the State Chamber of Commerce project. The Chamber of Commerce also helped pass the 1933 Riley Act, which imposed a minimum seismic requirement that applied to structures generally, not just public schools.

Consensus on further action proved elusive, however, and the code itself was not published until 1939. Even then, certain seismic design issues remained unresolved, so that there were two versions for lateral forces. The 1939 Chamber of Commerce code was never adopted by any public agency, but the example set by its use in regulating the seismic design of public schools significantly influenced California's engineering practice for the better.

BUILDING CODE FOR CALIFORNIA

PREPARED FOR THE

CALIFORNIA STATE CHAMBER OF COMMERCE

BY COMMITTEES REPRESENTING

Northern California Chapter, The American Institute of Architects
Southern California Chapter, The American Institute of Architects
State Association of California Architects
Northern California Sections, American Society of Civil Engineers
Southern California Sections, American Society of Civil Engineers
Southern California Chapter, Associated General Contractors of America
General Contractors of San Francisco

EDITOR

Edwin Bergstrom · Los Angeles

1939

Price Five Dollars

SECTION 3400

PART THIRTY-FOUR

STRUCTURAL SAFETY
GENERAL PROVISIONS
AND PROTECTION AGAINST EARTHQUAKES

SECTION 3400. METHODS OF STRUCTURAL DESIGN.

(a) *Loads, Stresses, and Methods of Design.* Every fire block, fire division, and building and every structural part thereof shall be designed in accordance with the loads, stresses and methods of design set forth in this Code that are applicable to the building under consideration. In the absence of definite provisions in this Code for the design of any fire block or building or structural part thereof, the method of design used therefor shall admit of analysis in accordance with the established principles of mechanics and of structural design, and be approved by the Board of Examiners and Appeal [*Section 400(e)*].

(a1) Every floor of every fire block in a building shall be designed to carry, without exceeding the design working stresses prescribed in this Code, the dead loads imposed on it and the gravity live loads due to the predominant purpose for which the floor is used, the minimum amounts of such live loads being prescribed in this Code.

Every roof and every appendage of a building shall be similarly designed, and the gravity live loads assumed to be carried by the appendages shall be those prescribed by this Code.

(a2) Every building and part and appendage thereof shall be designed to resist, at least to the extent required by this Code, the wind forces and lateral forces that are or may be imposed on it, the minimum amounts of such forces being prescribed in this Code.

(b) *Members and Elements Subject to Combined Direct and Flexural Stresses.* Structural members and elements subject to combined bending and direct stresses, with the maximum bending occurring at a point outside the middle third of the length of such member or element, shall be designed and proportioned so that the maximum combined unit working stresses in the end thirds of the member or element will not exceed the amount allowed in this Code for flexural unit working stresses, and so that said combined unit working stresses in the middle

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third of the member or element will not exceed the amount allowed for axial unit working stresses; *provided* that, if the flexural unit working stress does not exceed ten (10) per cent of the axial working stress, then no account of the flexural stress need be taken in the design.

(c) *Limiting Deflections.* The deflection of any beam, girder, joist, slab, or truss that is to support a plastered ceiling shall not exceed the number of inches or fractional part thereof prescribed by the following formula:

$$y = 0.1L^{\frac{7}{10}} \text{ ; wherein}$$

y = maximum deflection, in inches, and

L = clear span of beam, girder, joist, slab, or truss, in feet.

SECTION 3401. LATERAL FORCES.

(a) *Wind Force.* The wind pressure shall be considered to act inwardly or outwardly in any direction, upon the projection of the building or its appendage or roof structure on a vertical plane normal to the assumed direction of the wind.

(a1) The wind pressure assumed for any building not more than sixty (60) feet in height shall be not less than fifteen (15) pounds per square foot. If the height of the building is more than sixty (60) feet, then the wind pressure assumed for the portion of the building above said sixty feet shall be not less than twenty (20) pounds per square foot.

(a2) The wind pressure on tanks, smoke stacks, water cooling towers, signs, and similar exposed roof structures and their supports, shall be not less than twenty-five (25) pounds per square foot of gross area of the projected surface.

(a3) For combined wind and live load. the total vertical load on roofs need not be more than twenty (20) pounds per square foot, and the uplift pressure on flat or inclined roof surfaces shall be not less than ten (10) pounds per square foot of projected area.

(b) *Lateral Force Due to Earthquake.* The lateral forces due to earthquake shall be considered as acting in any horizontal direction, and the amount of such forces shall be as follows:

(b1) Wood framed buildings (Type W construction) shall resist a lateral force not less than *(Insert here the percentage set out in* *column 1 or column 2, paragraph (b1), of Appendix D)* of the combined dead and live loads required therefor by this Code.

(b2) *(Insert here the paragraph (b2) set out in column 1 or column 2 of Appendix D).*

(b3) Buildings having bearing walls of reinforced concrete or reinforced brick shall resist a lateral force equal to not less than *(Insert here the percentage set out in column 1 or column 2, paragraph (b3), of Appendix D)* of the combined dead and live loads required therefor by this Code.

(b4) Buildings having bearing walls of unreinforced masonry shall resist a lateral force equal to not less than ten (10) per cent of the combined dead and live loads required therefor by this Code.

(b5) Parapet walls, cantilever walls above roofs, exterior ornamentation, and appendages other than marquises, shall resist, normal to the wall, a lateral force equal to one hundred (100) per cent of their dead load weight. Roof structures, tank towers, tanks and contents, chimneys, smoke stacks, and marquises, shall resist a lateral force equal to twenty (20) per cent of the combined dead and live loads required therefor by this Code.

The values of lateral force given in this *sub-paragraph (b5)* shall not apply to the supporting structural members of the structures named therein, which need not resist greater lateral forces than those required for the entire structure.

(b6) For the combination of dead load, live load, and lateral forces, an increase of not more than *(Insert here the percentage set out in column 1 or column 2, paragraph (b6), of Appendix D)* of the unit working stresses required by this Code may be used in designing the strength of building members.

(c) *Live Load Basis for Lateral Force Design.* The live load basis that shall be used for the lateral force design of buildings and their fire-divisions to contain predominantly the kinds of occupancies listed in *column 6, Part 2, of Table 1202,* and for the lateral force design of the parts or appendages of buildings named in *column 2 of Table 1203,* shall be not less than the amount set out, in pounds per square foot, in *column 3, Part 2, of Table 1202,* nor less than the amount set out by reference, in pounds per square foot, or in pounds per lineal foot, in *column 6 of Table 1203,* respectively, in a box horizontally opposite

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the box containing the description of the kind of occupancy, or the part or appendage of the building under consideration.

(d) *Required Resistance Against Torsional Moments.* In buildings having members that will act as rigid horizontal diaphragms, the structural units which resist the lateral earthquake force shall be so arranged that, in any horizontal plane, the centroid of such resisting structural units will be coincident with the center of gravity of the weight of the building; otherwise proper provision shall be made for the resulting torsional moment of the building.

(e) *Distribution of Shears.* Shears shall be distributed to the various resisting units in accordance with the principle of relative rigidities.

(f) *Reducing Gravity Live Loads for Lateral Force Design.* Except as provided in *Section 3401(c),* unit gravity live loads [*Section 3402*] may be reduced twenty-five (25) per cent for lateral force design.

(g) *Loads for Retaining Walls.* The lateral pressure of earth, or other materials, including the effect of partial or complete saturation of earth and the effect of surcharge shall be computed in accordance with a formula approved by the Building Inspector, but in no case shall earth pressure on a vertical or approximately vertical wall, without any hydrostatic pressure, be taken less than a fluid pressure of twenty-five (25) pounds per square foot per foot of depth, plus the equivalent depth of surcharge. The surcharge for sidewalk loads shall be assumed not less than two (2) feet, and the surcharge for street loads shall be assumed not less than three (3) feet.

SECTION 3402. GRAVITY LIVE LOADS.

(a) *Gravity Live Loads Required.* The unit gravity live loads that shall be used in the design of any fire block to contain predominantly a kind of occupancy listed *in column 6, Part 2, of Table 1202,* or in the design of any part or appendage of any building named *in column 2 of Table 1203,* shall be not less than the amount set out as a concentrated amount, or by reference, or in pounds per square foot of horizontal projection of floor, roof, part, or appendage, or in pounds per lineal foot, *in column 2, Part 2, Table 1202* and *in column 5 of Table 1203,* respectively, in a box horizontally opposite the box containing the description of the kind of occupancy of the fire block or the name of the part or appendage or the building under consideration.

(b) *Unlisted Live Loads, and Live Loads for Unlisted Buildings.* Unit gravity live loads not listed in said *Table 1202 or Table 1203* shall be determined from the proposed use or occupancy, in the manner prescribed in *Section 502* of this Code, but no such gravity live load for a fire block used or occupied for storage, warehouse, or similar purpose shall be less than one hundred twenty-five (125) pounds per square foot.

(c) *Snow Loads.* If snow is anticipated, roofs shall be designed for the probable increase in loading, and the total snow load shall be used in lateral force design.

(d) *Partition Loads.* A partition load used in the design of floors may be considered either as a concentrated load or as a uniformly distributed load, equal, in pounds per square foot, to one-twelfth (1/12) of the weight of the partition per linear foot.

(e) *Arrangement of Live Loads.* If the gravity live load is less than one hundred (100) pounds per square foot, or is less than twice the dead load, the moment effect of partial loading on columns may be disregarded. The assumed arrangement of live loads for determining the maximum stresses to be resisted need not be more severe than that of simultaneously loading alternate panels of every floor, an arrangement of loading in which vertical tiers of loaded panels alternate with vertical tiers of unloaded panels.

(f) *Allowable Reduction of Live Loads.* Beams, girders, and trusses that support a tributary floor area in excess of one hundred fifty (150) square feet in area shall be proportioned to carry the full dead load, plus not less than eighty (80) per cent of the required gravity live loads supported thereby.

(g) All columns, piers, bearing walls, and bearing partitions shall be proportioned to carry not less than sixty (60) per cent of the gravity live loads supported thereby; provided, that no reduction shall be made in gravity live loads required on roofs when computing the loads carried by such columns, piers, bearing walls, and bearing partitions, and that no reduction in gravity live loads shall be made for such structural members of warehouses, library stack rooms, and other buildings or parts thereof used for storage purposes.

APPENDIX
D

APPENDIX D

AMOUNT OF LATERAL FORCES DUE TO EARTHQUAKE

The engineers and architects unanimously agree that the effects of lateral forces should be taken into account in the design of buildings to resist earthquakes. The amount of the forces that should be assumed for that purpose and the modifications of the unit working stresses that may be permitted in designing resistance to such forces are set out in *columns 1 and 2 of this Appendix D.* See *page 427.*

The municipality should adopt the provisions set out in one of the said two columns and write them into the proper sub-paragraphs of *Section 3401* of its Code, as follows:

1. The percentages set out opposite *(b1)* and *(b3)* in *column 1* or in *column 2* should be adopted by the municipality and inserted in the proper places in *sub-paragraphs (b1)* and *(b3), respectively, of Section 3401.*

2. The text opposite *(b2)* in *column 1* or in *column 2* should be adopted by the municipality and inserted as the text of *sub-paragraph (b2) of Section 3401.*

3. The maximum amount that working stresses may be increased for the combination of dead load, live load, and lateral force is set out as a percentage of the required working stress, opposite *(b6)* in *columns 1 and 2* below: the percentage set out in *column 1* may be adopted by the municipality if the amounts of the lateral forces set out in *column 1* are adopted by it, and inserted in the proper place in *sub-paragraph (b6) of Section 3401.* If the amounts of the lateral forces set out in *column 2* are adopted, then the percentage set out opposite *(b6)* in *column 2* should be adopted and inserted in the proper place in *sub-paragraph (b6) of Section 3401.*

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Column 1	Column 2
(b1) five *(5)* percent;	*(b1)* six *(6)* percent;
(b2) In buildings having structural frames, the columns and beams of such frames, together with slabs, walls, or other structural elements, and their connections, which may be constructed to act with the frames as distributing elements in resistance to lateral forces, shall be made capable of resisting a lateral force equal to not less than two (2) percent of the combined dead and live loads required by this Code, and the structure shall resist the lateral forces, expressed in percentages of the combined dead and live loads required therefor by this Code, as follows: Top two floors of building...8% 3rd and 4th floors from top of building..................6% 5th and 6th floors from top of building..................4% All floors below 6th floor from top of building...........2% Theatres and other buildings without regular floor levels shall resist a lateral load equal to five *(5)* percent of the combined dead and live loads required therefor by this Code.	*(b2)* Buildings in which the structural frames are designed to resist a lateral force equal to not less than two 2 percent of the combined dead and live loads, shall resist a lateral force equal to not less than six *(6)* percent of the combined dead and live loads required therefor by this Code. If the structural frames are designed to resist less than two *(2)* per cent of said loads, then the entire structures shall resist a lateral force equal to eight *(8)* percent of said load;
(b3) five *(5)* percent;	*(b3)* eight (8) percent;
(b6) seventy-five *(75)* percent.	*(b6)* thirty-three and one-third (33⅓) percent.

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CONNECTIONS

The EERI Oral History Series

John E. Rinne

Foreword

I interviewed John Rinne at his Kensington home in the San Francisco Bay area several times from 1986 through 1988, when he was nearing 80. Although he had been retired for several years at the time of the interviews, Rinne was still vigorous and active.

It was easy interviewing him because he had in mind a pretty good road map for topics he wanted to cover. At the time, I was focusing my oral history efforts primarily on seismic safety, and interviewed Rinne especially to cover the development of seismic design in northern California and his leadership of the Joint Committee. The discussion of *Separate 66*, an important chapter in the development of seismic design, occupies half of this entire oral history. If I had it to do over, I would ask more questions about some of Rinne's other substantial contributions, as well as more on his family, and his personal motivations and views on the practice of engineering.

John Rinne was born in San Francisco in 1909 to parents who had immigrated from Finland. He grew up in Albany (near Berkeley, California) and graduated from Berkeley High School. He attended the University of California at Berkeley and graduated Phi Beta Kappa in 1931. It was the height of the Depression, and work was sporadic and hard to get. In 1932 Rinne went back to U.C. Berkeley for his Masters degree, which he received in 1934. He then worked with many of the more forward-thinking engineers in practice in the San Francisco Bay Area at the time, such as John Huber, Henry Dewell, and Austin Earl. In 1937 he began his 32-year career at Chevron, where he soon became supervisor of the civil and architectural department. Rinne retired from Chevron in 1969 and joined Earl and Wright as a vice-president, spending much of his time supervising the design and construction of offshore platforms in the North Sea. He retired from Earl and Wright in 1980.

Throughout his career, John Rinne played a remarkable leadership role in earthquake engineering and in professional earthquake engineering organizations. He was active in EERI in its early days, and chaired the committee that set up the First World Conference on Earthquake Engineering. In 1948 he became chair of the Joint Committee and shepherded the design and code effort that resulted in the landmark publication of *Separate 66* in 1951 in the ASCE *Journal.* Rinne was president of the Structural Engineers Association of Northern California (SEAONC); president of the statewide Structural Engineers Association of California (SEAOC); president of EERI in 1966-1967; and second president of the International Association for Earthquake Engineering, an organization he helped to found. In 1973 he became president of the national American Society of Civil Engineers (ASCE).

John Rinne died October 16, 1992 at the age of 83, after a lifetime devoted to the engineering profession and the improvement of seismic design.

Stanley Scott
Research Associate and
Research Political Scientist, Retired
University of California, Berkeley
March 1996

A Personal Introduction

When you shook hands with John Rinne, you were instantly aware that he had known manual labor. I was proud of my own strong handshake, but he easily bested me, laughing all the while at my grip. I later learned from his younger brother Clarence that during his teens, John had worked at the White Lumberyard in Berkeley. Handling and stacking lumber all day long every day developed his large strong hands. An immigrant from Finland, John's father had raised his five children in the American tradition of hard work and instilled in them a desire for a university education.

According to Clarence Rinne, John always wanted to be an engineer, and aggressively pursued that career at U.C. Berkeley, where he graduated with high honors. Clarence had other ideas at first, and favored English and history, but switched to engineering because it offered a more promising future in those times of economic stress. All jobs were hard to get, however, in those Depression years, and after graduation John and Clarence followed the practice of all young engineers by going from office to office for employment. Where they found work depended on which office had been awarded a design contract.

One of the firms was Huber and Knapik Consulting Civil Engineers in San Francisco, where both John and Clarence worked at various times. This was in the early 1930s, when a growing controversy among civil engineers had emerged. Those who were doing structural design for buildings felt that the American Society of Civil Engineers (ASCE) was not responding to their special needs, particularly with respect to their fee schedules. This resulted in formation of regional structural engineers associations in California, and soon afterward of the statewide umbrella group called SEAOC, the Structural Engineers Association of California.

At the time, many of the structural engineers left ASCE altogether. Walter Huber of Huber and Knapik, and one of John Rinne's mentors, thought otherwise, resolving to stay with ASCE and its structural section. He never joined SEAOC, but went on to become national president of ASCE. John's approach was quite different, as he was active in both organizations, becoming SEAOC president in 1953, and national ASCE president in 1973.

I came to know John personally in 1952 when, with my boss and mentor Rube Binder of Bethlehem Steel, I made frequent trips from our Los Angeles office to San

Francisco. In 1953 I moved to the San Francisco Bay Area and transferred membership to the Structural Engineers Association of Northern California. Rube continued his liaison trips between Los Angeles and San Francisco. He introduced me to the leading northern California structural engineers of that time, including Henry Degenkolb, John Blume, Art Sedgwick, Henry Powers and Harold Hammill.

In both northern and southern California, engineers continued their activity on building code development, but were primarily concerned with strictly local problems, which were considered unique. John Rinne, however, clearly recognized the need for a statewide code, particularly as his company, Standard Oil, was building facilities in most areas of California. EERI's First World Conference on Earthquake Engineering, held in Berkeley in 1956, also helped ignite California interest in earthquake-resistant codes and stimulated further dialogue between the two areas, north and south.

These north-south discussions became more formal when southern California structural engineer Bill Wheeler was appointed chairman of a 16-member SEAOC seismology committee in the fall of 1956. Wheeler selected Rube Binder as his vice chair, liaison and special advisor. Rube also continued his shuttle diplomacy between the north and the south, helping to reconcile regional differences on seismic design policy in a three year process that produced the first SEAOC Blue Book, including the 1959 Recommendations.

I believe the continuing dialogue between Rube Binder and John Rinne had a great deal to do with the achievement of a north-south consensus. Small and excitable, Rube was a marked contrast to John's 6-foot-4-inch, 200 pound quiet presence. Both Rube and John, however, shared a devotion to their profession and a desire to seek common ground for agreement in engineering judgments.

John chaired a subcommittee set up to prepare the 1960 Commentary explaining the basis for the new SEAOC Recommendations. Roy Johnston and Herman Finch completed the subcommittee membership. I was active on the SEAONC seismology committee, and was assigned to help with the Commentary. I recall that while we all contributed as best we could, it was John who did the actual writing and editing of the Commentary drafts. I was very much impressed by the quality of his writing and his editorial skills--his original drafts did not need much revision.

We also had the benefit of John's extensive technical knowledge of all areas of civil engineering. Furthermore he brought to the task what he learned from his earlier experience in 1948-1951 as chairman of the ASCE/SEAONC joint committee that produced the ASCE document called

"Separate 66." *Separate 66* firmly established the dynamic basis for earthquake analysis of structures and building code design requirements. John's oral history describes the joint committee's handiwork in some detail, providing informative information to engineers seeking to learn how the present code developed.

As you read this oral history, you must come to realize that John Rinne was an extraordinary engineer, who rose to the top of his profession through skill, intelligence, and judgment, plus hard work and perseverance. He was also a fine human being, tolerant, kind, and thoughtful of others. I was privileged on occasion to be invited to his house, and enjoyed the warmth of his and Rose Marie's hospitality.

I can also remember him seated with his young sons on the sidelines of softball games at Silverado Country Club in the Napa Valley—before it became the exclusive resort it is now. SEAONC picnics held there were attended by virtually all members because of the opportunity it offered to meet informally and make friends with fellow engineers, while also enjoying sports events and a barbecue.

During 1993 memorial services for John, his son Ed had these insights into his father's character, given in a eulogy at their family church:

> *Dad loved ballgames, ice cream, a good story, nodding off when things got boring, singing, and a good game of cards. After achieving so much in his professional life, he finally took up golf when he was around 70, and managed to break into the 90's on a fairly regular basis while packing his clubs around Tilden. I never watched him lawn bowl but he certainly enjoyed it, and particularly the friendship at the club. His association with this church was particularly enjoyable, both spiritually and socially.*
>
> *Dad taught us as he did others, through the examples set in his actions, and was not one to lecture us much. In my case he was able to pass on some knack for civil engineering, and a straightforward approach to attacking and solving situations. But outside of his love and support, his high ethical standards and integrity stand out as qualities I will never forget.*

Robert Preece
Preece/Goudie & Associates
San Francisco
May 1996

Chapter 1

Background and Education

"I declared that I wanted to be a civil engineer, and selected a college preparatory curriculum, without knowing exactly what a civil engineer did."

Rinne: I was born in San Francisco, on September 24, 1909, of Finnish parents who came over here from Finland and who in 1915 became citizens of the United States. When I was very young—about three years old—my folks built a house in Albany, which is still occupied at 1035 Curtis Street. I went through the Berkeley school system, starting with Jefferson School at Rose and Sacramento streets, because nearby Marin School had not been built. Later, I went through the sixth grade at the Marin School.

I transferred to what is now the Martin Luther King Junior High School, which at that time was Garfield. From there I went to Berkeley High School, where I graduated in December, 1926. I worked at the lumberyard where my father was also employed, and had been for a number of years. I did heavy stevedoring work until summer 1927, at which time I went with a pal of mine, Louis Dragon, for a couple of weeks of vacation at the Berkeley City Camp at Echo Lake in the Sierras.

Then I started at Cal in the fall of 1927 and proceeded from there. Going back a way, it's interesting that when I was at

Garfield, going into the ninth grade, it was incumbent upon us to make a decision [tentative, perhaps] as to what our objective was. Were we going to take a college preparatory course at the high school, or prepare for a vocational, or a commercial type of curriculum? On my own, being the oldest member of our family of my generation, one of five children, I declared that I wanted to be a civil engineer, and selected a college preparatory curriculum, without knowing exactly what a civil engineer did. I had no civil engineering background, other than what one would read in the papers.

Scott: Something must have prompted your interest.

Rinne: Yes. It was largely due to an interest in mathematics. I was much more interested in that than in history, or English. Those subjects were not of my particular liking, although I did reasonably well in them. I graduated from Berkeley High School in December of 1926. I was one of the four commencement speakers, although I never would have recognized my own talk because it had been so heavily edited by my English teacher.

UC Berkeley: 1927-1931

Rinne: I won one of the $50 Kraft prizes in my freshman year at the University of California, Berkeley. It was strictly a scholarship type of thing. I had received very good grades. In fact, I did all the way through my four years at Cal to my Bachelors degree in 1931. I was a member of honor societies: Phi Beta Kappa, Tau Beta Pi, Chi Epsilon. As it turned out I was runner-up to the medalist.

Scott: Runner-up to the University Medalist—the University Medal is considered the highest or one of the highest of such awards given by the University?

Rinne: Scholarship awards, yes. In the summers, during vacation, I had worked for various companies. After my freshman year I had a stint at the surveying summer camp in Marin County. I put in a couple of months with Southern Pacific Company, digging post holes in the Sacramento Valley, which is enough to make one want to avoid that kind of labor for a living.

Later, in August 1934, I married my classmate Rose Marie Marcella Shiely, UCB 1931, who had immediately followed up her A.B. degree with a secondary teacher's credential. She was teaching commercial subjects in Sunnyvale and commuting to San Francisco/Berkeley so we could be together weekends. She was required to drop out of teaching when she got married. We had three sons, Stan in 1935, Ed in 1940, John M. in 1944.

Rose Marie passed away in 1974 in London, the result of an accident at home [Rinne was then located in London, working on North Sea oil projects]. I met up with Josephine Claussen in Berkeley—a Chevron widow and mother of a son, Dr. Bill Claussen, and daughter, Jane Trotman. Jo and I were married May 31, 1975. Both of us have grandchildren, she with five, I with four, several of whom are already pursuing postgraduate studies.

Summer Vacation Work at Chevron

Rinne: During my sophomore-junior-senior vacations I worked for Chevron at the Richmond refinery. To start with, Frank Maker was responsible for my getting a summer vacation job at Standard Oil of California's Richmond Refinery [now Chevron]. He was a next-door neighbor of a classmate of mine in high school, Warren Hoyt, and Warren introduced me to Frank. Frank took it upon himself to get me the summer job, following my sophomore year at U.C. Berkeley. It was not in engineering at the start. I was a helper in the company's boiler house at Richmond. About a month after I started that summer, the work in the boiler house slowed down and I, along with other vacation help, was transferred to the barrel house, and there we were engaged in loading boxcars with 42-gallon barrels of oil products.

In the following year, my junior year, during summer and Christmas vacations, I worked with the company's engineering department, which at that time was part of the Richmond Refinery work force for Chevron, which was then Standard Oil Company of California. The work of the engineering group at the Richmond Refinery involved mostly process plant design and materials ordering. Later, in 1937, when I started my 32 years with Chevron, the engineering department had been moved to San Francisco and existed as a separate general engineering department.

As I said, this was all largely at the encouragement of my good friend Frank L. Maker, who was an architect by training, but who was more of an engineer than an architect, because engineering was what he did most of the time—as a specialist in various branches of engineering, which an integrated oil company needed.

Scott: Maker was an employee of Chevron?

Rinne: Right. As it turned out, when I graduated in May of 1931 with my bachelors degree, Chevron had orders from its board of directors to hire no one. I was out on the streets for a bit.

Chapter 2

Employment During the Depression

"Ed Knapik, an associate of Walter L. Huber, asked me whether I wanted to go to work for them. Heavens! I was on the ferry boat the next morning."

Scott: When you graduated in 1931, the Depression was already severe?

Rinne: Yes, it had hit pretty hard. When I received my bachelor's degree in May of 1931, I had expected to be hired by Chevron. But company policy in the depth of the Depression ruled against any hires. That put me on a job hunt, which in June of 1931 landed me in Henry D. Dewell's office. Largely as a result of recommendations that my dad's boss gave to me, I was introduced, among others, to Henry Dewell—a structural engineer and an earthquake engineer of considerable renown in his day.

Perhaps with some reluctance, Dewell put me on and I went to work for him in June of 1931. Mainly I was working on the design of school buildings for Principia College in the Midwest. Henry Dewell's wife was a Christian Scientist, and Principia College was a strong Christian Science college. I guess she somehow had an influence on Dewell's getting the job of designing the structures for the Principia College. That involved the design of buildings back in Missouri, where earthquake was not a serious consideration.

I was also working on designs for the California Sugar Company. I did considerable shuttling back and forth between Dewell's office and Huber and Knapik's office. But we did also have some earthquake renovation work, principally for the C&H Sugar Refinery at Crockett, where they were doing work on strengthening some cast iron columns, among other things, and also looking at the lateral force capabilities of the old buildings there.

Work on Master's Degree

Rinne: Then in 1932 Dewell unfortunately suffered a stroke and virtually closed his office. Austin Earl, a University of California classmate of Henry Dewell, came in and with a skeleton crew continued to carry on those things that had to be done. Earl came in and took over Dewell's office as principal. He took over what work remained on the Principia job, where there were not that many buildings left to be done. It was more follow-up work of an engineering nature, following shop drawings and inspections. Dewell's office became Earl's office at that point. Buzz [Jonathan G.] Wright was there, and continued for a while, but then Buzz went to work on the foundations of the Golden Gate Bridge, only later to return to what became Earl and Wright. Dewell came back a year later, after he recovered from his stroke. While he was physically handicapped—his right side was inoperative—he was mentally alert. Then for a time the office was called Dewell and Earl, and later Earl and Wright.[1]

So I was let go because of Dewell's stroke, and left Dewell and Earl's office in 1932 to go back to school and work in Professor Raymond Davis's office and laboratory, while at the same time taking courses that led me to a master's degree in 1935. I worked in the materials laboratory under Professor Davis in his concrete research, related mostly to Hoover Dam. At the same time those of us who were working part time on the project were also taking some courses. Because I expected to work for Chevron eventually, I included courses in heat transfer, automotive engineering, advanced math, and vibrations. I completed the courses for my master's degree in one school year.

I did not get the M.S. until 1935, however, by which time I had completed the Master's thesis, which we were required to do at that time. It involved shrinkage studies on concrete blocks, which were cubes about 18 inches on a side. There was some testing involved, but the testing I was doing mostly related to my master's thesis. I was working for Davis more as a draftsman, along with my friend Phil Fletcher, a classmate of mine in 1931, who was a much better draftsman than I was, and which Raymond Davis recognized. He preferred Fletcher's drafting work to mine, and I do not blame him for that.

1. Henry Dewell opened his practice right after the Panama Pacific Exposition in 1915. He was an important figure in early-day earthquake engineering and was a leader on the team that worked on a state building code for California (the 1939 Chamber of Commerce code) after the 1925 Santa Barbara earthquake.

Shuttling Between Offices

Rinne: In 1933, while I was still at the University of California laboratory working on Professor Raymond Davis's Hoover Dam work, I received a call from Ed Knapik. Knapik, an associate of Walter L. Huber, asked me whether I wanted to go to work for them. Heavens! I was on the ferry boat the next morning to go to San Francisco to work for Huber and Knapik. It was work on the design of buildings—which involved buildings designed for lateral forces, albeit at that particular time the codification of school buildings was just getting under way. The Long Beach earthquake was in 1933.

I had worked for Huber and Knapik for about a year when their work load dropped down, whereas Dewell and Earl, as the firm was then called, had work to do on design of some structures for the Department of Water for the City of Sacramento. So I did design work on overhead storage tanks, and went up later [1936-1937] as a construction engineer on these facilities.

Scott: So you went back with Dewell—then called Dewell and Earl—and then worked in Sacramento?

Rinne: Yes, first in San Francisco on design, and in Sacramento for a year on engineering for construction. It was work for the City of Sacramento—actually for Dewell and Earl—on construction engineering work for water facilities we had designed in San Francisco in 1935-36. I went to Sacramento in 1936, a hot July day. It was 105 degrees in Sacramento at the time, I recall distinctly, and moving into a non-air-conditioned house wasn't exactly the most pleasant thing, but we soon learned how to accommodate ourselves to that kind of weather. When I came back from Sacramento in mid-1937, Dewell and Earl didn't have anything to do, but Huber and Knapik did, so I was back with Huber and Knapik.

Scott: How long had you been in Sacramento?

Rinne: Almost exactly a year. It was in July of 1937 that I came back, as I recall.

Recollections of Austin Earl and Henry Dewell

Scott: Would you give some more of your recollections of both Austin Earl and Henry Dewell. Earl apparently was a person of considerable stature in engineering back in those days. As you suggested, Dewell was recognized as an important figure in early-day earthquake engineering in the Bay Area.

Rinne: I first ran into Austin Earl when I went back to Dewell and Earl and worked on what developed as the Sacramento project I went up on in 1936. At that time, Earl was a partner of Henry Dewell—in Dewell and Earl—and he was an extremely good engineer. A very capable guy and he was also a crotchety old guy. Not necessarily hard to get along with, but kind of rough.

I remember, for example, when I was up in Sacramento on that water project, Dewell and Earl came up to Sacramento to inspect the overhead storage tank I was working on. I was up there as senior project engineer and had several people working for me up there, one of whom was a man older than me, and who had considerable experience in construction engineering. I was up high on the tank, which was 100 feet off the ground at the bottom, and then went on up another 40 feet or thereabouts. Anyway I was up on the tank, and Earl called up to me—in effect they wanted to start back home, to San Francisco. I said, "I'll be down in just a minute," or some words to that effect.

Later, I got a call from Henry Dewell to come to San Francisco. So, I was down there the next morning and got chewed out because I shouldn't have been so rude to a man of Earl's stature, to have said something that might have been mistaken as talking down to my superior. That was the extent of that particular deal. He had said something to Dewell. It did ire me a little bit, so my response might have raised hell and caused a little bit of a stink. I was concerned about it, because I told Henry Dewell that, here I was up there as the head of the construction and had older people working for me and had to have a little bit of respect paid the other way, too. Henry took the position that I should be careful how I should do this, how I handled Earl.

Earl was also responsible for the engineering of the Posey Tube across to Alameda, the first tube. He was also very active in the drilling of the Broadway Tunnel to Orinda, which ran into considerable trouble. Earl was a consultant on that, and he told the contractor how to do this work without having it cave in. They were having trouble because of ground movement there. The soil would have a tendency to move into the hole that had been excavated, so it was important to make sure they kept their lining work immediately behind the excavation—not to let the excavation get way ahead of the lining, which would permit a larger cave-in.

Walter Huber

Scott: When you were working with Dewell, did he promote the importance of seismic resistance, or instill in you a sense of the importance of seismic design?

Rinne: Well, of course for that I also have to credit Walter Huber, because jobs I had with Walter Huber at that time directly involved design for earthquake resistance. Some were school buildings. Both men [Huber and Dewell] recognized that earthquake forces on any structure related to masses [weights] and stiffnesses. In due course Huber achieved some national stature in the engineering profession, and became national president of the American Society of Civil Engineers [ASCE] in 1953.

Scott: Huber's seismic work on schools would have been after passage of the 1933 Field Act, which applied seismic standards to public schools?

Rinne: Yes. Shortly after the California legislature passed the Field Act, it also passed the Riley Act, for general construction in California, requiring a minimum 2 percent lateral force weight factor, leaving anything more than that up to the designer, the structural engineer, to do what he felt was necessary. The codes were quite variable at that time. Back then, San Francisco didn't have much of a seismic code at all, but primarily relied on resistance to wind forces, which relate to building face area exposed to the wind.

Scott: You mentioned Henry Dewell being active in early earthquake engineering and seismic design efforts, at least up to the time he had the stroke. In the late 1920s and early

1930s he did a lot of work on drafting preliminary versions of what became the building code published in 1939 by the California State Chamber of Commerce.

Rinne: Austin Earl was as active in earthquake engineering as Henry Dewell was. Earl was responsible for writing the earthquake provisions for the Uniform Building Code. I previously thought this work had followed the 1933 earthquake of Long Beach, but in March 1988 Buzz Wright assured me that it was done after the 1925 Santa Barbara earthquake. Earl was a good, straightforward, very succinct writer. He wrote good reports.

Chapter 3

Career

"I was responsible for establishing earthquake design criteria for Chevron's engineering department.... Later the criteria were influenced by, if not dictated by, the work of the Joint Committee."

Rinne: On returning from Sacramento in 1937, I again started working for Huber and Knapik. Then when I had worked for Huber and Knapik for 2 to 3 months, I got an invitation to go to work for Standard Oil of California, later Chevron, from Jim Stirton, who was assistant chief engineer for Chevron in the corporation's engineering department in San Francisco. The offer was good—paying all of $300 a month, a lot of money at that time, although it makes you laugh today. Walter Huber advised me to take the job, which I did.

[When I started my long-term employment with Chevron] I immediately got involved in the civil and architectural division of the engineering department, and shortly thereafter became supervisor of that division—a job I held for many of the 32 years that I worked for Chevron.

Scott: You moved up pretty fast?

Rinne: I moved up to supervisor of that particular division rather quickly. This involved all of the structural and civil engineering work, as well as the architectural work, that is demanded of a corporate engineering department. We had four or five architects working, as well as a group of half a dozen civil and structural engineers. We were involved in designing structures for vertical load requirements, but also for the lateral load requirements of earthquake as well as wind.

Among the structures other than buildings and refinery-type structures were offshore structures for the production of crude oil in the Santa Barbara and Los Angeles areas. Besides buildings and structures, the civil and architectural division was responsible for the design and construction of pipelines, which activity took me to Canada and Alaska in 1943 on a series of war-induced pipelines.

[When I started at Chevron in 1937] the engineering department was providing engineering functions for all of the operating departments, not only the manufacturing department responsible for the refineries. Nevertheless, a large proportion of the work the engineering department did was in relation to process plant design, for which I, as a civil engineer and structural engineer, had responsibility for the foundation designs and the structural aspects of support of vessels, for example. This included both wind design and earthquake design for lateral forces.

My first assignment on coming back to the engineering department, however, was one of doubling the size of the Bahrain Refinery from 10,000 barrels per day to 20,000 barrels per day. It was an already built 10,000 barrel-a-day refinery, and they expanded it to 20,000 barrels. It primarily involved work on the furnaces, the oil heaters, and on pumps and heat exchangers.

Then I got involved in civil engineering and architectural work. Not very long after I joined the engineering department in 1937, perhaps a year later as I recall, I became the supervisor of what was called the civil and architectural division, and the drafting room also, as the drafting room was constituted then—later the drafting room became a separate section of the engineering department. That happened within a year or a year and a half of my joining Chevron in 1937.

Carrying over my experience with Dewell and Earl and Huber and Knapik, where we were working on earthquake design of buildings, I was responsible for establishing earthquake design criteria for Chevron's engineering department and applying these criteria. Later the criteria were influenced by, if not dictated by, the work of the Joint Committee formed about 1948, and which I chaired. I will discuss both the Joint Committee and the report later.

1937-1969: Many Projects

Rinne: During my 32 years of service there at Chevron—1937-1969—the work included project management of many projects, including many major buildings in San Francisco at 225 Bush, then 555 Market Street, and many other areas of the company's operations. Included in the civil and architectural division's involvement were several pipeline projects. These included the World War II conversion of the PG&E's STANPAC gas line, which runs from Kettleman Hills to Los Medanos, and which normally even today supplies a good part of the gas that we burn in our [San Francisco-Oakland] Bay Area houses.

During wartime, however, it became incumbent upon us to convert that line to oil service, in order not to have to rely upon tankers running up the coast, subject then to possible Japanese submarines. I was construction engineer on that particular job. It involved construction not only on the work of conversion of the STANPAC pipeline, but also the addition of other pipelines that brought gas from the Rio Vista area in the San Joaquin Delta area to supplement or replace the gases that were lost due to taking the STANPAC line out of gas service.

The Canol Project

Rinne: In 1942 my pipeline experience also took me to Edmonton, Alberta, and north into Alaska and the Yukon Territory on the Canol pipeline. This was in the design and construction of several hundred miles of pipelines to provide crude oil from Norman Wells in the MacKenzie River area, to Whitehorse, Yukon Territory, where a new refinery was built to provide an alternative source of vitally needed gasolines to avoid sea transport, which was potentially interruptible by the Japanese. It also involved gasoline distribution pipelines ranging from Watson Lake in the territory, through Carcross and Whitehorse to Fairbanks, Alaska, and also Skagway to Carcross. This was the Canol Project. The Bechtel people were primarily interested in it as a joint effort called Bechtel-Price-Callahan.[2, 3]

Gilsonite Slurry Pipeline, Utah-Colorado

Rinne: A rather unique pipeline was the one for gilsonite [a form of carbon] slurry, for which the civil and architectural division had project responsibility. The pipeline ran from a gilsonite area in Utah to Colorado, where gilsonite was the feedstock for a new refinery there. The gilsonite was sent as a slurry because it was almost like coal, but was lighter and could be ground up, mixed with water into a slurry, and sent over the hill to the pipeline junction or the terminal near Grand Junction, Colorado. I think in recent years Chevron has sold that refinery.

Gulf Coast Refinery, Pascagula

Rinne: I was also a member of a team evaluating sites for what became a large Chevron refinery in Pascagula, Mississippi. That was an interesting assignment, technically as well as climatologically, including the mosquitoes that were in abundance down there. I might mention that in the summer mosquitoes were also abundant in Alaska and the Yukon Territory. We had about a six-man team down there, and the Pascagula people put us up in a facility, which was interesting. It was a nice place to be, but I personally made the recommendation that we not speak among ourselves at all, because at the time we were considering alternative sites for this refinery and I felt sure that the room was bugged. I am sure it was bugged. If we had given any indication at all of what we

2. Finnie, Richard, *Canol, The Sub-Arctic Pipeline and Refinery Project.* Constructed by Bechtel-Price-Callahan for the Corps of Engineers, United States Army, 1942-44. San Francisco, Ryder and Ingram, Publishers, 1945.
3. Ueda, Herbert T., D. E. Garfield, and F. D. Haynes, *The Canol Pipeline Project: A Historical Review.* Cold Regions Research and Engineering Laboratory, Special Report 77-34, Hanover, N. H., October 1977.

were interested in, they [the eavesdroppers] would have found out.

Anyway, we ended up selecting the Pascagula site on which the refinery was built. It has been expanded considerably since, both technically from the standpoint of being more flexible to be able to process almost any kind of a crude oil, and also enlarged. I do not know what its capacity is today, but it's probably in the order of 200,000 barrels a day crude feed rate.

Scott: Were the various sites you were considering within the Pascagula region?

Rinne: No, they included some sites over in Georgia. We looked at several different areas besides the Pascagula area, but we ended up there. It was all right. It turned out to be a good site. We had to build a large amount of storage tankage at Pascagula. The tankage was built on a foundation we knew was subject to settlement under the tanks. Foundation investigations indicated that a fairly uniform thickness of compressible soils overlay firmer soils below.

Before the Pascagula refinery project, largely as a result of recommendations originally made by the soil mechanics firm Dames and Moore, we had already adopted the idea of building the bottom of such a tank so that it could accommodate settlement, perhaps in time requiring releveling or recontouring of the bottom. That was much cheaper than building piled foundations to support the tanks. We saved the company a lot of money. The conditions at the Pascagula site, however, as well as at some other sites, were so uniform that there was no tendency for unequal settlement around the tank. It all went down together evenly, and there was not a big settlement on one side of the tank and only a little settlement on another side.

Offshore Platforms

Rinne: During the last part of my work for Chevron, while I was still heading the civil and architectural division, I was also a project manager for the offshore platforms that Chevron needed at that time. The first was Platform Hazel, which was built off of Santa Barbara, California. We later did other platforms in the Santa Barbara area, and down south of Los Angeles we had other kinds of offshore facilities for which I was responsible.

In short, before I left Chevron in 1969, I spent several years on offshore structure design, which included project management of projects like Platform Hazel and Platform Hilda, and a couple of other platforms that were done before I left Chevron. They were designed primarily for wave forces, and only incidentally checked for what now is considered to be a rather nominal lateral force system, although the wave forces themselves were responsible for substantial lateral forces against the structure.

The structures themselves were supported off the bottom. In the case of Hazel, they were supported on what would now be termed "spread footings." The caissons actually were jetted down to a firm-enough soil so that they didn't have supporting piles. Hazel was in about 100-foot water depth. Hilda, the next one, was in slightly deeper water, maybe 130 or 140 feet, and it did require piles, which were drilled and driven. In the case of Hilda, we did the design work ourselves in our division of Chevron.

Then in 1969 I retired from Chevron and went to work as a vice president of Earl and Wright [in San Francisco], at that time a subsidiary of SEDCO,, an offshore drilling company, where for ten years I worked on the design of offshore platforms, including a two-year stint in England [1974-1976], where I was managing director of a North Sea joint enterprise with C.J. Brown [CJB-Earl and Wright], an English contractor.

Working Again for Earl and Wright

Rinne: When I went to work for Earl and Wright in 1969, it had been purchased by SEDCO, an offshore drilling contractor. For a number of years, starting with the design of Platform Hazel off Santa Barbara, Earl and Wright had been working primarily on the design of offshore drilling platforms for the oil industry. Although Austin Earl had died several years earlier, the name Earl and Wright had been maintained from when I had been with Earl and Wright before [in 1932].

I joined Earl and Wright as a vice-president, and devoted ten years with them to managing offshore design projects, as well as developing new platform concepts, both for clients and for ourselves. New concepts were needed because the water depths of offshore work were increasing significantly. It has gotten to the point where now we think nothing of building a platform in 1000 feet of water depth. I say "We think nothing." Well, yes, we do think something—in fact, we think a lot!—but it is done.

During the ten years with Earl and Wright, I was in London for two years, 1974-76 as managing director of CJB-Earl and Wright, a joint venture primarily involving design of offshore platforms for the North Sea. Continuing in this same capacity when I returned from London, I retired from Earl and Wright on January 1, 1980. Except for an occasional consulting job on some buildings, since 1980 I have essentially been fully retired.

Chapter 4

Activities in Engineering Organizations

"I'm not so sure, however, that we know so much about proper and adequate design for earthquakes...."

Engineering Associations and Clubs

Rinne: About the time I went to Chevron in 1937, Ed Knapik was secretary/treasurer of the San Francisco Section, American Society of Civil Engineers (ASCE). Sometime after that, Knapik decided that he wanted to get out of the secretary/treasurer job, and convinced me that I should take it. So I became secretary/treasurer of the San Francisco Section. That was an important start to advancement through the years, to president, nationally, of the American Society of Civil Engineers.

During this period with Chevron I was active in professional societies—state, national, and international—concerned with seismic design and construction. I was involved both technically—e.g., chairing the Joint Committee of the San Francisco Section of ASCE and the Structural Engineers Association of Northern California (SEAONC)—and administratively—e.g., president and board member of ASCE locally [1954] and nationally [1973]. I was president of SEAONC in 1951; of the

Structural Engineers Association of California in 1953; of EERI, the Earthquake Engineering Research Institute, in 1966-67; and of the International Association for Earthquake Engineering (IAEE) in 1964-68.

Also at about the same time I joined Chevron I also joined the Engineers Club, and was a member of it for a number of years. I also joined SEAONC. Henry Dewell was not only a good structural engineer and a good earthquake engineer, but was also a member of the state registration board for civil engineering, and he induced me to grade papers on the civil engineering examination, which I did. As soon as I was eligible I took the civil engineering examination myself, in the structural engineering option—as it was at that time—and passed it readily. It wasn't difficult; I'd had enough experience to do that.

I received my civil engineers license when I was 25. About two years later, when I had enough years experience, I applied for and received a structural license. I then got involved on committees of the Structural Engineers Association of California. My work on codifying lateral forces began early, but I do not remember precisely what we were working on at that time. It was not until considerably later [1948] that the Joint Committee was formed that was responsible for producing the *Separate 66* report.

Joint Committee and *Separate 66*

Rinne: In 1948 I was appointed chairman of the Joint Committee of the San Francisco Section of ASCE and SEAONC. The Joint Committee's main objective was to formulate an earthquake lateral force code that we could recommend to the City of San Francisco.[4]

Earthquake Engineering Research Institute (EERI)

Rinne: While the Joint Committee was active, I was invited [in 1951] to join the Earthquake Engineering Research Institute. It was then a very exclusive group, providing consultation to the U.S. Coast and Geodetic Survey on earthquake engineering, and especially on measurement of earthquake strong motion. Much later, EERI opened its membership so that it has become a national organization. In fact, one of the more recent presidents has been Professor Bob Whitman from MIT. In the earlier years it was led mostly by California engineers. I was also president of EERI for a relatively short time [1966-1967], and before that was on the EERI board of directors.

Rinne: In 1955, as a member of the EERI board I suggested that we initiate a world conference on earthquake engineering. This was undertaken and held at the University of California, in July of 1956. I was general chairman of that conference, which included papers from many foreign countries, including Japan, Chile, New Zealand, and Peru.

The second world conference was held in Japan in 1962, and we have continued to hold these world earthquake conferences on a quadrennial basis, starting in 1956. It is amazing that the

4. In 1948 San Francisco adopted the "Vensano" lateral force code provisions, devised by Harry Vensano, then director of public works for the City. Vensano's code was controversial in the San Francisco engineering community. Adoption of the Vensano code prompted formation of the Joint Committee to work on what came to be known as the "*Separate 66*" report, and which recommended new lateral force design provisions.

first issue of proceedings of the 1956 conference was a single volume about an inch thick, whereas our later earthquake engineering world conferences produce several volumes.

International Association for Earthquake Engineering (IAEE)

Rinne: As a result of the first two conferences—the one in Berkeley to start with [1956], and the next one in Japan [1960], the Japanese suggested that we establish an International Association for Earthquake Engineering (IAEE). They would sponsor the headquarters office and assume the expenses of handling that office. They have done that right up to the present, and have done an excellent job. I was suggested as a vice president of IAEE, and Kyioshi Muto was its first president. Then I became president of IAEE, serving between the New Zealand conference in 1965 and the Chile conference in 1968. I might add that more and more countries have joined the International Association for Earthquake Engineering. I'm trying to remember—the latest was Nationalist China.

Scott: Will you comment a little about the work of the international association—is holding the quadrennial world conferences one of its main purposes?

Rinne: Yes, the world conference is the principal purpose of the IAEE. Under Japanese leadership, the IAEE has been very good about publishing books to disseminate information, such as world compilations of codes from various countries. They've come out with those periodically, as well as with seismological bulletins. The code compilation from 1988 includes code provisions for earthquake resistance from 36 countries. There have also been other sponsors of reports of earthquakes, outside of these organizations IAEE and EERI, that have made significant contributions to earthquake engineering. Other publications have been sponsored by steel companies, the American Institute of Steel Construction, and the concrete people through the American Concrete Institute. Also by the Portland Cement Association and their laboratory in Illinois.

In all, from the standpoint of correspondence and dissemination of information, I would say we have had a large amount of information published. We certainly know a lot about earthquakes. I'm not so sure, however, that we know so much about proper and adequate design for earthquakes, despite all the papers. That's one thing that gives me considerable concern, because even the most modern-designed buildings still have to stand the test of a major earthquake, and that's liable to come at any time. Our experience to date hasn't been all that good.[5]

5. These observations were made several years before the 1994 Northridge earthquake revealed significant damage in many modern steel frame buildings.

Chapter 5

Developing a New Design Code

"We wanted to take a fresh approach to developing a code."

Scott: You have mentioned the Joint Committee that produced the report called *Separate 66*, which many regard as something of a landmark. I hope you will discuss the Joint Committee and its work in some detail. But first, could you set the stage by saying a little about your recollections of the development of seismic codes and earthquake engineering design, going back to your early years of practice?

Early Seismic Codes

Rinne: The Long Beach earthquake occurred in 1933, and at the time I was employed with Huber and Knapik. We were aware of what that earthquake did to buildings down there. We were designing buildings such as Washington High School in San Francisco and the Caswell Coffee Building, where we did design for lateral forces, as much as 10 percent, which was high for that particular time. For the Caswell Coffee building it worked out rather easily.

In the case of school buildings, shortly after the Long Beach earthquake [under the Field Act] the first Appendix A for public school buildings came out. There again we were applying Appendix A, as it was called, the earthquake criteria, that the state stipulated under the Field Act for the design of public school buildings. Initially, those were rather modest factors, 8 percent or 10 percent at the maximum. It was no big problem [to incorporate those criteria] in the building configurations that they had at that time.

Scott: Please discuss the Caswell Coffee Building a little more.

Rinne: Yes. Back in 1934, one of the jobs I had with Huber and Knapik was on the Caswell Coffee Building, located on Rincon Hill, San Francisco. It is a flat slab building with outside bearing walls—a formidable building, practically windowless. I designed it for lateral forces of 10 percent gravity, and Walter Huber said, when he found out what I'd done, "That's okay—that's probably a good factor, but it's more than required by code." He went back to the architect and convinced the architect that the building was inherently good for large lateral forces. Designing it for the 10 percent g was easy from the design and construction standpoint.

Scott: You say the building was virtually windowless. A building like that would probably have a good deal of inherent strength, would it not, that is if it had a decent basic structure at all? So was a virtually windowless structure with 10 percent for lateral forces considered a pretty substantial design, in terms of earthquake resistance?

Rinne: It was substantial for that time, but not by modern standards, because that kind of building now goes up as high as 16 percent. But I have no fear about the Caswell Coffee Building sustaining an earthquake rather readily.

Scott: Say a little about why you designed the Caswell Building the way you did. That was in the early days of trying to design for seismic forces.

Rinne: It was designed after the Long Beach earthquake in 1933, which caused a lot more concern [about seismic design]. The 1933 earthquake was important because there was a fairly large lateral force, greater than wind, and it was the first earthquake that had a ground accelerograph recording of the motion. The accelerograph recorded a lateral force acceleration maximum of about 35 percent of gravity in the ground. That does not mean that the structure responded to 35 percent, but it does mean that the peak of the short-period motion—the high-frequency motion—did have that high a ground acceleration. This kind of evidence of such strong lateral force motion concerned all of the structural engineers, even in the absence of codes. Codes got imposed rather quickly after that. I'd have to go back to the record for the timing of the codes as to when the codes of the 1930s and 1940s came out.

Scott: When you talk about codes, are you talking about local codes, or the Uniform Building Code, or both? I recollect comments about seismic inclusion in codes somewhere in the late '20s or early '30s. I believe the Santa Barbara earthquake in 1925 aroused a good deal of interest, and there was a great deal more interest after the Long Beach earthquake.

Rinne: The first Uniform Building Code (UBC) came out following the Santa Barbara earthquake, and according to "Buzz" Wright, to whom I spoke in March 1988, it was written by A.W. Earl. Of course, the 1906 earthquake did a pretty good job of damaging a large area of northern California, but San Francisco was very reluctant to do anything about earthquake codes, in spite of all the damage the earthquake did. I'm amazed at how reluctant they were—but they were.

Formation of the Joint Committee

Rinne: Finally, because of the influence in the Los Angeles area and the south generally, the Uniform Building Code and the Vensano code [in 1948] got to be a bit excessive, in the opinion of San Francisco engineers in defining design lateral forces that were considerably higher than those in other codes. That's one of the concerns that emphasized the need to get an up-to-date code in San Francisco, which prompted the formation of the Joint Committee.

We wanted to take a fresh approach to developing a code. The codes were quite variable at that time, the 1930s and 1940s. Los Angeles was developing a seismic code, but San Francisco didn't have much of a code at all until adoption of the Vensano code in 1948.

Scott: You are now referring specifically to codes that include some earthquake provisions?

Rinne: Yes, any code that included earthquake-resistance factors and criteria. The International Conference of Building Officials (ICBO) had done quite a bit at that time, too, influenced by work of the Seismology Committee of the statewide association SEAOC. There was considerable variation in the codes, and San Francisco was in need of a revised code. This is what initiated the formation of the Joint Committee in 1948. It was also the awareness of various other codes that were being practiced in California, and a feeling that San Francisco engineers ought to take a look at the matter, independent of what had gone on before. That was our charge. The Joint Committee was made up entirely of practicing engineers, in contrast to people who were analytical academicians.

Scott: It was set up as a joint committee of the northern California civil engineers and structural engineers. Did SEAONC provide the primary push, or was it also the civil engineers?

Rinne: It came from the two organizations. Both the structural engineers [SEAONC] and the civil engineers (San Francisco Section, ASCE) appointed the Joint Committee in 1948. The Joint Committee's meetings started shortly after it was appointed. We met weekly for dinner and a "skull session" following, at the El Jardin restaurant on California Street near Market Street, San Francisco. I remember the room so well.

Criteria and Definitions

Rinne: With the help of my associates Ed Robison and Milton Ludwig, and with the spectral response curves of Professor Maurice Biot at Caltech as I mention later, I think I contributed as much as anyone to going in the direction of defining the design base shear, and from the base shear, defining what the forces were on the structure, and the resulting overturning moments. I was influenced also by my own understanding, coming from my vibration course in my M.S. year at Berkeley.

We on the Joint Committee were looking for a new approach. I think I, and my cohorts Ed Robison and Milton Ludwig, espoused the code represented by *Separate 66* and later codes. That was a new idea at the time—as far as codes were concerned—the direction it took, and still takes, and the method of defining the seismic forces to be used in the design of buildings, primarily, with some criteria also included for "other structures."

Scott: You are referring to the direction taken by the Joint Committee's deliberations?

Rinne: Yes, the path taken toward defining the seismic forces. The Joint Committee worked on a definition of earthquake-resistance criteria, based fundamentally on determining a design base shear, in contrast to specifying forces directly. Since the earthquake forces come into a structure through the ground, it was logical to go from the ground up. It was also consistent with vibration theory.

Scott: It was a system for dealing with lateral forces?

Rinne: It was more a system for defining the forces. From the base shear you eventually got to the forces, and from the forces to the shears and the moments, to provide your design criteria. The earthquake has its input from the ground up, and we were starting to get ground motion criteria. The 1940 El Centro earthquake record has been used extensively in analyses. We got the first strong ground motion record in Long Beach in 1933. But the El Centro record was a more recent, clearer, and more complete record, that is often quoted and used. Of course we have had many other records since then.

Work of Biot, Robison, Ludwig

Rinne: Anyway, we came up with the criteria that are contained in *Separate 66*, as a result of many weekly meetings of the committee. We wrestled through this effort and were making use of work done by Maurice A. Biot, a professor of mathematics at Caltech. Biot worked on a theory of elastic systems vibrating, with an application to earthquake-resistant buildings. That was his first work, in 1933,[7] and then he went on to analyze a number of others and came up with this idea of a spectral response—a vibration response of the structure to the earthquake.

Scott: Biot was developing a theoretical way to help understand how a building vibrates in response to earthquake forces?

Rinne: Yes, the way a building vibrates in response to the ground motion. Biot did that early work in 1933, and more work later to generate both undamped and damped spectral responses of simple one-mass structures. It wasn't until 1948-1950 that Ed Robison did his work, confirming Biot's type of work and coming up with spectral responses that were comparable. Also there was work by Milton Ludwig. Through electrical analogies—Ludwig was an electrical engineer—he provided this idea of equivalent one-mass systems to represent the fundamental second, third modes in both a flexural-type structure and a shear-type structure. This also came out of Ed Robison's work in analyzing the many modes (vibration

7. Biot, Maurice A., "Theory of Elastic Systems Vibrating Under Transient Impulse with an Application to Earthquake-Proof Buildings," *Proceedings of the National Academy of Sciences,* Vol. 10, No. 2, February 1933.

shapes) in which a multistory (hence multi-mass) building could vibrate, and each mode could be represented by a one-mass system plus a vibration shape.

Biot published the first spectral response of single degree of freedom (SDF) structures to earthquake ground motion—the so-called Duhamel equation—published in the Bulletin of the Seismological Society of America, as I recall, around 1949 or a little earlier [it was 1941]. This came out in the early stages of the Joint Committee's deliberations, and preceded Ed Robison's work.[8]

Edward C. Robison was a classmate of mine at U.C. Berkeley, graduating in 1931. Ed worked with the U.S. Coast and Geodetic Survey and was primarily responsible for the design of the first strong motion accelerograph, which was constructed and set up in time to record the Long Beach earthquake of 1933. The instrument is still a museum piece on display at major technical meetings to show what they had to do in those days to get a record. That should have been credited to the work of Ed Robison, who was working in the Coast and Geodetic Survey at the time.[9]

8. Two of Biot's publications from the early 1940s were footnoted in the *Separate 66* report: 1.) Biot, Maurice A., "A Mechanical Analyzer for the Prediction of Earthquake Stresses, *SSA Bulletin*, Vol. 31, No. 2, April 1941. The method was developed at Caltech in 1932 [Biot, 1933], and in Biot's words, was an attempt to draw "a curve representing some kind of harmonic analysis of an earthquake, where the acceleration intensity is plotted as a function of frequency." 2.) M.A. Biot, "Analytical and Experimental Methods in Engineering Seismology," *Transactions*, Vol. 108, ASCE 1943.

In 1948-1950, before the powerful computers became available, Ed Robison used logarithms to analyze the vibration characteristics of a 15-mass building, and his work was very helpful in visualizing the multi-mode action of a vibrating structure. It established a principle stated in current publications that the total mass of a building (or other structure) can be assigned to the translational modes in which it can vibrate under the impetus of the ground motion. Ed made the extended calculation by longhand, using seven-place logarithms. He developed a matrix that indicated the distribution of the building weight into the first 12 of 15 modes that it could vibrate in, indicating that the sum of these modal weights added up to the weights of each story, and in total added up to the total weight of the building. This was something we were saying was true at the time we did *Separate 66*. Ed's work was very helpful to us.

In his calculations, Ed used the Alexander Building analysis, which had been worked on by John Blume and Harry Hesselmeyer as a master's thesis at Stanford in the mid-1930s.

9. Both the published record and the recollection of Ralph S. McLean, who worked with Robison prior to the Long Beach earthquake, suggest that Robison did not design the instrument. While it appears that Robison definitely had a role in the development and assembly of the recorders that were installed before the Long Beach earthquake, the actual designer evidently was Frank Wenner of the Bureau of Standards, see N.H. Heck, H.E. McComb, and F.P. Ulrich, "Strong Motion Program and Tiltmeters," in *Earthquake Investigations in California: 1934-1935*, Coast and Geodetic Survey, Publication No. 201, pp. 5, 7, 1936. Oral history interviews were conducted with McLean in 1990-1991, and an exchange of correspondence in 1995 dealt specifically with the instrument-design question.

Based on rigidity, weights, and foundation flexibility, Blume and Hesselmeyer calculated and reconciled the 1.25 second measured fundamental period of the building with the calculated period.[10]

The late Milton Ludwig of Chevron, a co-worker of mine, also developed the concept for uniform "rods," in shear, and, separately, moment types of deflections. Ludwig contributed much to the earthquake analysis method that the Joint Committee put forward in *Separate 66*. It was the first time the earthquake design approach used the response to ground motion as a base shear—as it should be, since the base is where the vibration originates—rather than defining the lateral forces on the structure's masses empirically as forces acting at the floor levels of the building.

Scott: So all three of them contributed to those developments, starting with Biot and the concept of spectral response?

Rinne: That is correct.

Design Requirements and Observations of Performance

Rinne: We made the distinction that we were using Biot's work qualitatively. To translate Biot's data to quantitative design factors for lateral forces, we felt it was important that we make comparisons or make observations of damaged and nondamaged buildings in earthquakes. Henry Degenkolb was one of the foremost of the people who has observed the performances of buildings in earthquakes, and he compared these [observations] with what they were calculated to be able to resist. Incidentally, some of the buildings that stood up well in the 1906 earthquake in San Francisco, even by generous allowances, would hardly have withstood 2 percent g as a lateral force factor. Where we're at now is something else again, up to 16.7 percent g for low, rigid buildings.

Scott: You are referring to buildings that actually stood up in the 1906 earthquake?

Rinne: Yes, like the Flood Building.

Scott: But reasoning by conventional design theory, they should not have stood up to the lateral forces of the earthquake as well as they did?

Rinne: Yes, they should have sustained a lot more damage than they actually did. On the other hand, in more recent years we've designed structures to much more rigorous requirements, as related to forces, and have found ourselves in a jackpot because in many cases the [more modern] buildings have not performed very well, for various reasons. This is not to say that some old buildings are not bad—there are bad old buildings. In some of the developing countries, where they are very limited in the materials available and the mate-

10. The 1950 Robison material was published by EERI. See John A. Blume and John E. Rinne, eds., "Vibration Characteristics and Earthquake Forces in a Fifteen Story Building—An Abridgment of An Original Paper by Edward C. Robison, 1950," *Earthquake Spectra*. Vol. 5, No. 4, EERI, November, 1989. The brief biographical sketch of Robison included in the *Spectra* article also alludes to the Long Beach accelerograph design and Robison's role: "He, apparently, was responsible for the design—at least the mathematics of the design—of the strong motion accelerograph that recorded the first strong motion record in Long Beach, California, in 1933."

rials have obviously been very, very poor, they had serious collapses, such as earlier in Nicaragua, China, and Russia, and in San Salvador more recently.

Emphasis on Practical Design Criteria

Rinne: As I say, because the *Separate 66* Committee was a group of practicing engineers, we put heavy emphasis on what practical design criteria should be. And that established a range of coefficients for the base shear, which varied from the 2 percent up to 6 percent, with the maximum of 6 percent for buildings of 0.25-second fundamental period, and the minimum of 2 percent applying to buildings with fundamental periods of 0.7 of a second or longer.

Scott: So the requirement related to the fundamental vibration period of the structure.

Rinne: Yes, that was the result of the work of the committee. A.K. Chopra's *Dynamics of Structures: A Primer*, published by EERI in 1981,[11] for example, on page 115, indicates a wide disparity between the theoretical response spectrum and the code-related spectra. It is this disparity that is significant in the code design criteria for earthquake resistance. These spectra, incidentally, are based on 5 percent damping, which is probably reasonable.

Scott: That is for spectral response analysis, based on information on motion in an actual earthquake? The graph compares actual ground motion with the code provisions?

11. Chopra, Anil K., *Dynamics of Structures: A Primer*, EERI, 1981.

Rinne: Yes, the El Centro ground motion, for example.

Code and Response Spectra Disparities

Scott: Would you comment on the implications of the wide disparity between the code figures and the theoretical response spectra?

Rinne: Part of the difference of analytical spectral response is based on 5 percent damping, which is a convenient mathematical device. Also, of course, the objective of earthquake resistance is fundamentally to save lives, and secondarily to avoid major damage. In any case, I do not believe that we are in a position to analyze buildings to the extent of assuring—except in very rare cases—that there will be absolutely no damage at all under any circumstances. The imposition of very conservative criteria certainly is justified for nuclear power plants, but generally, I think that would be asking too much. We can, however, provide buildings that have the basic stability required for earthquake-resistance, although receiving perhaps some damage.

An earthquake does not last forever. Earthquakes are not generally as long as the Alaska earthquake, which is reported to have lasted maybe 4 minutes. And duration significantly influences damage to many kinds of structures. The extent to which they respond in a fundamental mode, and the extent to which they respond in a higher mode, is indicated by the response curve. In other words, the fundamental period may be out where the response coefficient is well below peak, but applied to a major proportion of the total weight of the structure. The second and perhaps third modes might be at the peak spectral response, but multiplied by the significantly lower weights assigned to higher modes.

Chapter 6

Separate 66 Report

"I don't even remember how I became chairman, but I did, and it was kind of interesting."

Scott: Say something about the committee that produced the *Separate 66* report. How was it set up, how were the members selected, and how did you work with it as chairman? What were you trying to accomplish and how did you go about reaching a consensus? How did the process work?

Rinne: It was surprisingly good. There was very little resistance to taking on some rather drastic changes in concept, the result of my study of the work of Biot, Robison, Ludwig, and others.

Scott: Had you done that work—your study of Biot and others—quite a bit earlier, perhaps over a number of years, or did you concentrate on it at the time, after you knew you would be doing a lot on the *Separate 66*?

Rinne: I was doing it largely as a result of interest I already had prior to the formation of the committee. But it was not until we got into the committee work that I suggested we might use the concept advocated or suggested by Biot, and others too. Surprisingly enough, the Joint Committee liked the thought of taking a fresh look at it, without any commitment.

The committee was appointed by the two societies, as noted earlier. I do not remember just how they arrived at who was going to be on it, and who was to be chairman. As a matter of

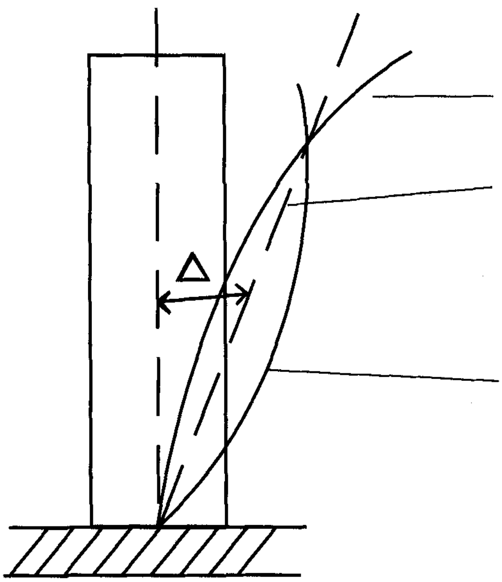


Figure A

fact, I don't even remember how I became chairman, but I did, and it was kind of interesting. The fact that we were given a complete free rein to take a completely independent look at the earthquake code in light of the concepts and technology available at the time, was in itself interesting.

Shear Distribution

Rinne: I remember distinctly the evening when we were talking about how we should distribute the shear vertically. I came up with the figure [Figure A] here in the *Separate 66*. This shows that under circumstances of shear, or an average between a shear and a moment-type of building (a pretty high building), the distribution would be pretty much a straight line, triangular. That brought us to the triangular distribution. I remember going to the blackboard, and putting that formula on the board. The formula continues in force-equivalent code provisions to this day. Measurements of actual accelerations in multistory buildings during earthquakes in years following the Joint Committee report do a good job of confirming the reasonableness of the triangular distributions formula.

Scott: You say that there was surprisingly little resistance to the concept of defining base shear, and the distribution of the base shear on the structure as design lateral forces?

Rinne: Yes. At that time I remember my saying I would have had a minimum of 2 percent, and a maximum of 8 percent rather than a maximum of 6 percent. But some committee members were willing to go from a 2 percent minimum to a 4 percent maximum. So we compromised at 2 percent to 6 percent, as the lateral force coefficient range for the base shear.

Scott: You say that you yourself would have gone for the more conservative figures, a range of from 2 percent to 8 percent?

Rinne: Yes, because of the nature of spectral responses that were available back then, even in Biot's time. Actually, we are now much higher [more conservative] than that, in some respects. With the passage of time and technology developments, code provisions have been changed to higher factors, rather than to lower factors. Offsetting the higher base shears and forces to a significant amount has been the use of higher strength materials: structural steel, reinforcing steel, and concrete. Also modular analysis, permitted readily with the aid of computers and computer programs, has provided better analyses, as the current codes now recognize.

Report Preparation and Publication

Scott: How long did it take for the committee to arrive at this consensus?

Rinne: In one evening at the El Jardin we concluded the base shear range [after several months of consideration]. It escapes me entirely as to who actually did the writing of the report. I think I did as much as anybody, but I do not remember which parts I wrote, and which I did not. I'm certainly responsible for a good part of it. The chairman frequently gets that job.

An interesting aside here [Rinne quotes from *Separate 66*]: "The behavior of structures in earthquakes has generally been recognized as a dynamic vibration phenomenon of a transient nature. Although rigorous solutions are possible for particular ground motions applied to particular structures, these solutions are too involved and of too limited significance to be of direct practical value to the structural engineer. The more rigorous methods, however, should be encouraged to guide their thinking toward less rigorous but more practical methods." Those look like my words. It sounds like me. As it is now, we are quite capable of making elaborate analyses if someone will pay the costs—in time and dollars. Computers have helped enormously in making complicated analyses practical. Most offices now use them.

Scott: How long did the work on *Separate 66* take?

Rinne: We actually met for a couple of years before we got to writing the report. I think we had also actually submitted our report earlier than 1951, as we probably reported to our two local associations in 1950, but I do not recall the exact dates. Then it took 6 months to a year just to get it published. The Joint Committee report and recommendations were originally published in the April, 1951 Proceedings—Separate No. 66, and since then have frequently been referred to simply as "*Separate 66.*" It was also published in 1952, complete with discussions, in volume 117 of the Transactions of the American Society of Civil Engineers.[12] In 1953, the Joint Committee received the ASCE Moisseiff Award for its report.

12. Anderson, Arthur W., John A. Blume, et al., "Lateral Forces of Earthquake and Wind," *Separate 66, Journal of the Structural Division*, Proceedings of the American Society of Civil Engineers, ASCE, New York, NY, 1951. (Also "Lateral Forces of Earthquake and Wind," *Transactions of the American Society of Civil Engineers*, Vol. 117. ASCE, New York, NY, 1952.)

Acceptance and Use

Scott: Could you talk a little about how you hoped the report would be used, and how it was actually used, especially in the first years afterwards. How influential was it? I guess you were trying to change both the codes and the practice of the profession.

Rinne: The acceptance of this by the societies was one thing. But its actual use as a code by any of the political organizations—cities or whatever—came somewhat later and in modified form. Following this, there was the Seismology Committee of the Structural Engineers Association of California, of which I was a member for many years. That committee started with this [*Separate 66*] as a basis. They modified the Joint Committee code somewhat, and also sponsored its recommendations to some extent with the ICBO and the Uniform Building Code, and also with the municipalities for use in local codes. I'm not familiar with exactly what San Francisco has now as a code. I've not had occasion to do any design in San Francisco, and I do not have their current code. I know what the Uniform Building Code looks like, and it is an adaptation of *Separate 66* into more modern conservative formats, adopting modular analyses, and resulting from analyses made of responses to particular earthquakes and observations in earthquakes.

Scott: Observations in earthquakes like Bakersfield, San Fernando, and Alaska?

Rinne: Those, plus earthquakes in foreign lands including Mexico [1985], Nicaragua [1972], China [1976], etc. Well, things have been done to eliminate what we called a J-factor—a reduction factor applied to the overturning moment of prescribed forces, which first made its appearance in the early SEAOC code. To a significant extent the J-factor effect is reintroduced by the acceptance of modular analysis and combining modular responses by root-sum-square additions. Overturning moment is slightly reduced in buildings over about 12 stories designed by so-called static equivalent forces. The National Earthquake Hazard Reduction Program (NEHRP), of the Building Seismic Safety Council, says that additionally the foundation overturning moment can be reduced 10 percent. That is the first time I've seen that kind of reduction. In buildings taller than 12 stories, moments are slightly reduced down to the foundation. The foundation overturning moment can be further reduced by this 10 percent.

On Overturning: A Little Optimistic

Rinne: I think I was basically a little optimistic in early SEAOC work in our definition of the J-factor. But at the same time, I have a feeling that a large part of the throwing out of the J-factor was due to the inadequacy of concept of the buildings—which caused extremely high shears and overturning moments in columns (especially corner columns), inadequate design of the columns, and inadequate ability of columns to take high compression without shear failures, especially in concrete. That's been indicated by a need for much more rigorous stirrups in reinforced concrete columns than has been the practice before; also more utilization of spiral reinforcement. Thoroughly confining the concrete inside the spiral is much more effective than stirrups. At least as indicated by the Olive View Hospital [San Fernando, California], the spiral concept is a very good concept for confining concrete so that it does not fall out when shaken by an earthquake, even if it may crack. Of course it does not do much for the concrete that is outside the spirals. That falls off in nothing flat.

Scott: You also mentioned stirrups. What are they and how do they work?

Rinne: Yes. Stirrups are spaced at intervals in the upper column and in beams and girders. They reinforce for shear stress in concrete columns—really for tension stress associated with shear in beams and girders. But at the same time they're spaced so far apart that concrete can fall out, causing column collapse

Scott: Stirrups are less effective?

Rinne: Yes. The spiral is much more effective, because it typically has a spacing of say 2 inches, and it retains the concrete intact, whereas if you have stirrups spaced at 8 inches, the concrete can fail, fall out and ultimately cause column collapse. So I think columns, such as those that are typical in the first story of reinforced concrete buildings, should either have spiral reinforcement, or else should have stirrups spaced in the same order of magnitude, spaced 2 to 3 inches apart, all the way up the column. After you get above the first story, it might be less, but I'm not sure of that. It depends on what other lateral-force resistances you have.

J-Factor

Scott: Going back to *Separate 66*—was the J-factor something that was put in *Separate 66* for the first time? I've never fully understood the J-factor, although Henry Degenkolb has talked about it a number of times. Was it a feature of *Separate 66*?

Rinne: No. *Separate 66* merely states that overturning moment be constant below 10 stories from the top of higher rise buildings. This was a bit optimistic.

Scott: Since *Separate 66* was written, there no doubt have been quite a few changes in seismic code criteria generally accepted in California?

Rinne: Yes, there have been important changes—and almost without exception they have been toward more conservative code criteria. Also in recognizing our abilities with the modern computers to make more elaborate modal analyses, including response in the fundamental mode and higher modes as needed. The J-factor, answering your question, was a factor reducing the overturning moment from that resulting from assigning all of the lateral force to the fundamental mode (or triangular distribution) to account for the distribution to fundamental plus higher modes. Higher modes add little to overturning in the lower stories.

Scott: When you say more conservative, you mean higher values?

Rinne: Higher values in general, yes.

Scott: As I understand it, *Separate 66* got picked up in the Uniform Building Code, although with modification.

Rinne: Yes, with conservative modifications.

Scott: Was this done fairly soon?

Rinne: No, it took a long time. They were not in any big hurry. Their procedures were changing, and that takes a while. You just do not do it hurriedly.

Back to the question about the J-factor being in the *Separate 66*. It is not, but it was included in item 8, p. 746 of the 1952 ASCE Transactions for future study: "establishment of design criteria for the overturning effects in earthquakes based on dynamic considerations." It must have been in later work that the J-factor itself turned up, including the work in establishing the Blue Books, the recommendations of the Structural Engineers Association of California.

Scott: But it was not referred to in *Separate 66*?

Rinne: No, in *Separate 66* it was not referred to as a J-factor.

Scott: You mentioned the SEAOC Blue Book. Say something about the relationship between the two reports—was *Separate 66* a forerunner of the Blue Book?

Rinne: Yes, *Separate 66* was a forerunner of the work of the SEAOC Seismology Committee. There has now been a whole series of Blue Book editions. I have the original ones, but I think I have them in boxes somewhere. The one I have here is dated 1967.[13] There is some history in here in the Blue Book, however, that is significant: the history of earthquake codes in California. These are the recommendations and commentary of the Seismology Committee of the Structural Engineers Association of California. In this 1967 edition they have C = .05 over the cube root of T, the fundamental period, to establish the design base shear by V = CW.

13. *Recommended Lateral Force Requirements and Commentary*, SEAOC Seismology Committee, Sacramento, CA. 1st edition, 1959; published periodically.

Scott: There seems to be a little discussion here of problems they have had dealing with overturning, the changes in the J-factor.

Rinne: They are saying that it was not until 1948 that San Francisco had anything more stringent than the Riley Act in its code. The table of variable coefficients was adopted, with a maximum value for one story of 8 percent and minimum value for 30 stories of 3.7 percent. Incidentally, that has variations for soil conditions. These were applied to design vertical loads, but those are coefficients that Harry Vensano—San Francisco's director of public works—was basically responsible for, and he personally was responsible for San Francisco adopting that particular code. That action of Harry's, he was a forthright guy, he was a little hard-nosed too, but his one-handedness prompted the formation in 1948 of the Joint Committee.

In 1956 San Francisco adopted a variation of the recommendations of the Joint Committee, in which the maximum [design coefficient] was 7.5 percent and minimum was 3.5 percent. That isn't an awful lot different from Harry Vensano's earlier figures of 8 percent and 3.7 percent. So he did reduce it a little bit, but that was 1956.

Scott: Reading between the lines suggests to me that it was a remarkable accomplishment on Harry Vensano's part to get San Francisco to do that when he did back in 1948.

Rinne: Yes, he was the responsible city official. This is the 1973 edition of the Blue Book, and there is a rather brief history here—it is something that you should have available. You can see that the code has developed through the years. It would be interesting to take all of them and see the changes in those fundamental things, of defining the maximum-minimum values of C, the formula for C, the formula or equivalent for J.

Hindsight Critique of *Separate 66* Recommendations

Rinne: The report of the Joint Committee included a number of recommendations for future studies. I would like now to go over these, to see how far we have progressed in the last 40 years [1948-1988], in expanding our knowledge and understanding of ground motions and earthquake effects, and also to indicate the need for improvements, over and above what was recommended in the text of *Separate 66*. The report made eleven recommendations for future studies. I will review each recommendation individually, in sequence.

Number 1: More Ground Motion Records

Rinne: Recommendation Number 1 pointed to the need for more extensive and active records of ground motions. This objective has been accomplished to a large extent, and has introduced the ground motion considerations into codes in various ways. One is as an S-factor for soils (which is also indicated in Recommendation Number 4). Ground motion is also a factor in a ratio—a response ratio to maximum ground acceleration. This has been accomplished in the latest code proposals, one being the tentative SEAOC 1987 code, and the other being the NEHRP 1986 code.

Scott: The first recommendation emphasized the importance of ground motion records, and their use in seismic code drafting?

Rinne: Yes, ground motions, and the fact that the committee talked about the accelerations. All I'm saying is that work has been done on this, and it is being accommodated in the codes in one way or another. A little later, in going through the other recommendations, we'll discuss some other aspects of the response question. Nowadays we have arrays of strong motion earthquake instrumentation—some on the ground, some in buildings and other structures. This is giving us a lot of information that we did not have when the 1951 Joint Committee report was written. Back then we were using the work that Professor Biot did at Caltech, and also work that Ed Robison and Milt Ludwig did at that time. So we recognized the need for more data related to earthquake motions.

Scott: Back in those days did you consider the lack of hard, concrete information on earthquake ground motion to have been a major gap in knowledge?

Rinne: Yes. Since earthquakes work from the ground up, it is logical that there is a strong need to understand the nature of ground motion in an earthquake. The motion isn't always the same, obviously—one earthquake isn't exactly like another earthquake, particularly because of differences due to location and seismicity. Nevertheless, there are some similarities, and these are reflected in analyses made of the ground motion, as simplified structures would respond to it. This is the way it enters into our consideration today—as an input factor to establish what we're calling "design spectra."

Number 2: Response Curves of One-Mass Systems

Rinne: Number 2 urged that we obtain additional response information for one-mass systems—that is what these response curves are. In the intervening years, a large amount of work has been done for one-mass systems, including the effects of damping and inelastic response. Again, this relates very closely to ground motion. The establishment of these spectra is one of the ways, with the aid of computers, of calculating the response of real structures to ground motions.

Scott: You said "one-mass" systems?

Rinne: Yes. The response curves are for the responses of a single-mass system of varying period. That's what a response curve is. One problem that comes up relative to codes in this regard is, again, the fact that no two earthquakes are exactly the same. Also, to a considerable extent, the responses of structures depend upon distance from the fault line. Earthquake motion close to the fault line includes a high degree of high-frequency, high-intensity ground motion. The high-frequency motion is damped out, however, as you proceed away from the fault line, whereas the lower-frequency, longer-period vibrations can travel much farther, and in fact become a very definite hazard to taller structures, as experienced in the Alaska earthquake of 1964.

Number 3: Building Vibration Periods

Rinne: Recommendation Number 3 was for a more complete examination of the periods of vibration of buildings, and of factors affecting such periods. The criteria in *Separate 66* for estimating period admittedly were very inaccurate, but at the same time were very conservative. Because the period is an important function (actually the abscissa) for determining the response, any inaccuracy might shift your design one way or the other rather importantly. It happens that if the period estimate is on the low side, you design for higher lateral forces than if you had a more accurate and higher assessment of period, and proceeded down the curve to a lesser force on the structure's base shear. Today there are recognized methods of calculating the period more accurately. The problem, really, is to be able to calculate before you complete the structural design. The designer can check the period calculation after completing the design. In earlier stages it's going to be something of an approximation, because you do not know exactly what the structure is going to be.

Number 4: Effects of Soil Conditions

Rinne: Recommendation Number 4 relates to some of the conditions I noted earlier in talking about ground motion. The S-factor was used in some of the earlier codes, including UBC. Generally higher amplitudes of ground motion are expected in weaker soil. A foundation on hard rock would have several characteristics, one being a lesser ground motion. Thus you should have less-severe motion of the ground in hard rock. On the other hand, there is another aspect of soil conditions in which the soil-structure interaction has come into play in recent years.

That involves flexibility of the soils. The overturning effect in an earthquake causes a rotation at the base of a building in softer soils. In some cases this can actually improve the response of the structure, because it lengthens the period and gets it farther out on the response curve. So despite softer soils having larger amplitudes of motion, they also provide this kind of flexibility, which is not usually taken into account. In codes it is frequently specified that the period calculation shall be made with the base being considered fixed or inflexible.

Number 5: Model and Shaking-Table Studies

Rinne: Recommendation Number 5 urged that additional model and shaking-table studies be done. We have gone a considerable distance in that, with the fairly large models and shaking tables now available, including the University of California facility in Richmond. These studies have not only added to our understanding of the responses of structures, but also have corroborated the analytical findings. In addition a lot of progress has been made in structural detail analysis and in structural materials performance.

Number 6: Lateral Force Design Criteria

Rinne: Recommendation Number 6 calls for a study of the rigidity criteria for the distribution of design lateral forces of various types of structures. We have also progressed in this field. I recall that the Japanese, who commented, and also Caltech people who discussed *Separate 66*, were somewhat critical of our so-called "triangular" distribution of the base shear. The reason they were critical was that they felt in very tall structures we get more motion or even higher-mode motion, which would increase the lateral forces in the top stories of a building. That was taken into consideration as early as some of the UBC codes, based upon work largely done by the Structural Engineers Association of California. This has been done in the UBC code, for example, by adding a certain percentage of the base shear as a lateral force at the top, and distributing the remainder of the base shear in a triangular fashion, represented by the formulas which have appeared in *Separate 66*, and also in later codes.

Number 7: Allowable Stress Increases for Short-term Loading

Rinne: Recommendation Number 7 was for further study of allowable increases in working stresses for short-time lateral loading combined with normal vertical loading, and noting the then-current practice of a 33 percent increase. So far there has not been any great change in this, except for the fact that we are using an entirely different philosophy in design from just a simple 33 percent increase on elastic allowable stresses. In other words, we are now using load factors, combined with higher ultimate strength and yield strength, that are somewhat different. In essence we agreed that for the earthquake contingency we should design using stresses somewhat higher than the normal stresses used with normally applied loads.

Number 8: Design Criteria for Overturning

Rinne: Recommendation Number 8 is for the establishment of design criteria for overturning effects based on dynamic considerations. We have come full circle on that. In *Separate 66*, as I remember, we considered the overturning moment only for the top 10 stories, or the top 120 feet of other structures, and assumed that the overturning moment would be constant below that.

Since we are now dealing with buildings that are 40, 50, 60, or 100 stories high, those criteria are no longer very valid. And at the same time we were doing more involved dynamic modal calculations, which indicate that the overturning moment is not solely dependent upon the shear (which is attributable to the fundamental mode of vibration), but that other modes come in that have a lesser effect on the overturning effect. In the early stage of the SEAOC criteria, we had a J-factor which was too low as defined by $J = 0.5/T2/3$, where T is the fundamental mode period.[14]

The net result was that the UBC and SEAOC both eliminated the J-factor entirely, requiring the application of the full set of lateral forces applied in triangular distribution down to the foundation. In more recent years there have been some steps toward decreasing the overturning moment and that has been somewhat arbitrary also, as it is in the NEHRP code of 1986. But in the NEHRP code for dynamic modal design, proper assignment of lateral force is given to both the fundamental mode and the higher modes, in accordance with portions of the mass of the structure that respond in each particular mode. In analyzing the 15-mass building that Ed Robison worked on, the J-factor equivalent obtained by combining modal spectral responses by root-sum-square is 0.686, compared with $J = 0.431$ by the $J = 0.5/T2/3$.

14. Rinne, John E., "Design Criteria for Shear and Overturning Moment," *Proceedings*, Second World Conference on Earthquake Engineering, Vol. III, 1960.

Scott: For clarification, when you say 15 mass, is that equivalent to 15 stories?

Rinne: Yes, 15 stories above street level floor. In the calculation it has been considered 15 masses, second floor to roof inclusive. I was going to say that in this particular building, the response in the fundamental mode involved about 75 percent of the mass, and about 20 percent responded to the second mode, 5 percent to the third mode, and negligible amounts in modes above that. The overturning effect even of the second mode was very small. Consequently, in effect you had the equivalent of the J-factor of 68 percent for that particular building. There is still some question as to what that factor should be, but in the dynamic analyses that we can now make, with the aid of computers and the response data available, it turns out that some reduction in the overturning moment definitely is in order, especially in taller structures.

Scott: The inclusion of the J-factor, as well as its value if included, relate to whether you reduce the projected overturning moment, and if so by how much?

Rinne: That's right, the J-factor is a multiplying factor—that generally is less than unity.

Number 9: Cataloging Design Data on Buildings

Rinne: Recommendation Number 9 merely calls for the cataloging of buildings or structures being designed for seismic forces, collecting basic data so that in the event of a major earthquake, early reports of damage and its relationship to design and codes can be compiled. With the advent of electronic instruments that are relatively cheap and portable, there has been a lot of instrumentation of buildings and structures. In fact, there are requirements to put strong motion instruments on the roof level, the foundation level, and intermediate levels, in taller structures.

After an earthquake, we are in a position to acquire a lot of information on what the forces were in the building, the actual forces felt during the earthquake. The accelerations are actually what you measure, and relate that to what design criteria were used. Many such records pretty well confirm the triangular distribution of forces up the height of multistory buildings.

Scott: After each earthquake we now get a lot more data on the motion. But I wonder whether buildings are being cataloged with respect to their design criteria.

Rinne: I have not seen anything actually done on cataloging, except perhaps in design engineers' offices, done because they would want to know what happened to their buildings—how they performed in an earthquake.

Number 10: Wind Effects on Low Buildings

Rinne: Recommendation Number 10 is for further study of wind effects on low buildings. Normally, we considered wind and earthquake together. *Separate 66* was devoted almost exclusively to earthquake forces, and just mentioned wind incidentally because wind is the other lateral force that we have to consider in design. Considering wind and earthquake forces jointly is probably in order. What has happened regarding wind effects on low buildings is that we have a lesser wind force compared to earthquake forces. California conditions were considered to be different than those in many other parts of the country, but then we do not have hurricanes.

Scott: Are you saying that we have a lesser force for wind stipulated in California than elsewhere?

Rinne: For wind, yes.

Number 11: Earthquake Effects on Bridges

Rinne: Recommendation Number 11 was for studies of earthquake effects on bridges. Bridges were not included in *Separate 66*, which was devoted entirely to buildings. We dealt with minimum earthquake forces for buildings, and bridges are really a separate topic. They are not subject to building codes. Bridges are related to highways, and we have little impact on what their criteria are.

I do not even know what criteria they are currently using for bridges, but I can fully appreciate that the kind of criteria used for bridges should recognize isolated foundations, in contrast to building foundations, which are normally tied together, so that the building as a whole has to move. In a building, one foundation should not move differently laterally than another foundation. In the past, that kind of movement has caused considerable damage in bridges. Bridge studies have been made, but in California at least, this has not been done by SEAOC.

Significant Progress Since

Rinne: That has fairly well covered the *Separate 66* recommendations for future studies. To sum up, we have made very significant progress in the application of seismic design knowledge in the intervening years from 1951 to the present. Now, before going on to the criticisms of *Separate 66* published in the *Transactions*, I would like to discuss this graph of design spectra, the progress we have made, and the direction in which we have been moving (see Figure B, Code Modal Design Spectra).

Scott: These are code modal design spectra curves that you have drawn on this graph [referring to Figure B].

Rinne: Yes. Through the years we have moved to higher force factors, from the time of the Riley Act, represented by the line near the bottom of the graph. This next curve was the Joint Committee's *Separate 66* recommendation. Here we have the SEAOC code from 1959 to 1973—there have been big changes for a number of years. And here are the spectra for hard rock foundation conditions, by SEAOC. This one by NEHRP is also for good soil conditions. You can see that even currently there are still some differences of opinion. But the spectra curves do indicate increasingly more demanding criteria as we learn from earthquakes.

Substantial Later Changes

Scott: Substantial changes appear to have been made over the years.

Rinne: Yes, there have been very substantial changes. I made this graph for comparative purposes only. For this purpose I've assumed a certain factor that enters into some of the codes that are not 1, but may be less than or more than 1, are not incorporated in this. The Joint Committee code did not have any factors called Z, I, K, nor did it relate to ground motion S factors at all. So, in making this comparison I assume that these were 1. Now in California Z would be 1.

The factor I relates to the importance of the structure, which might increase the lateral force from what is indicated here. K is a factor that does not apply to a dynamic modal analysis, but does apply to the UBC type of design. K is a factor that is anywhere from 1.33 on the high side to 0.67 on the low side. So, using K = 1 here is merely for comparative purposes.

Codes prior to the dynamic criteria alternatives are all based upon defining the base shear and establishing the "equivalent static forces" on the structure as a whole by distribution of the base shear. When spectra are used for dynamic analysis, they are applied to only a portion of the structural weight assigned to each significant mode. The modal responses are added in some such fashion as the square root of the sum of the modal response squares. For example, if you had a 1.25-second period structure, and you had designed by NEHRP spectrum, the factor here is 0.07 applied as the first mode, and the second mode period is let's say 0.5 of a second and its response up here would be 0.135. First of all, this 0.07 would be applied to 75 percent of the weight of the building, and this 0.135 would be applied to 25 percent of the weight of the building. Thus, while the factors are higher, they are applied to lesser weight.

So much for that. I am getting into technical details that I shouldn't be getting into. But the important thing to realize is that we have increased the [design criteria for] response of structures, especially in the shorter period. When we get to the longer periods, proportionately they seem to be getting closer together.

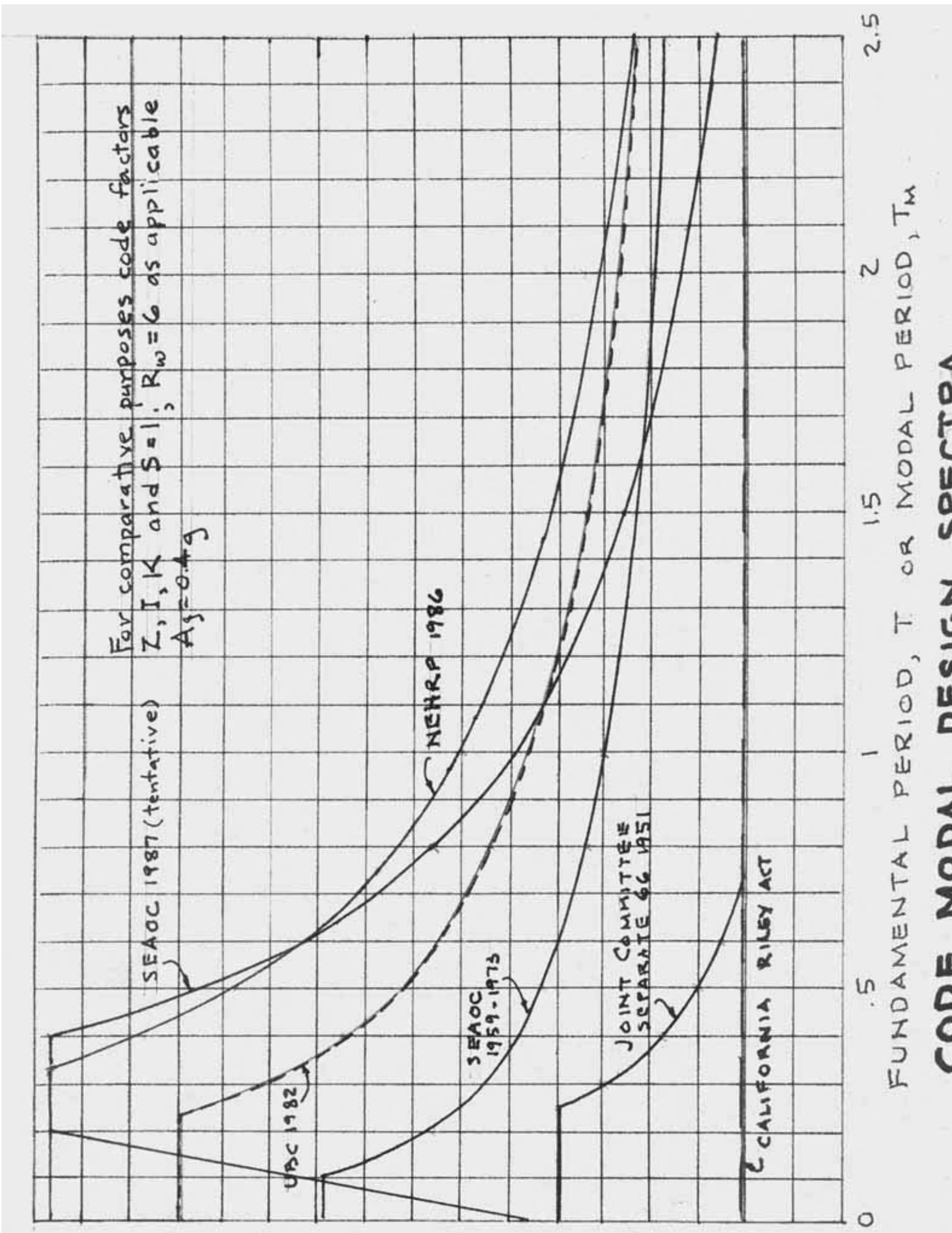

CODE MODAL DESIGN SPECTRA
FUNDAMENTAL PERIOD, T OR MODAL PERIOD, T_M
BASE SHEAR COEFFICIENT C or $C\frac{I}{R_w}$
For comparative purposes code factors
Z, I, K and S = 1., R_w = 6 as applicable
A_j = 0.4 g
SEAOC 1987 (tentative)
NEHRP 1986
UBC 1982
SEAOC 1959-1973
JOINT COMMITTEE SEPARATE 66 1951
CALIFORNIA RILEY ACT
0
.5
1
1.5
2
2.5
0
.02
.04
.06
.08
.10
.12
.14
.16

Figure B

Modes of Vibration

Scott: When you talk about the different modes, are you referring to the different natural modes and frequencies in which a multi-mass structure is capable of vibration modes?

Rinne: Yes. A fixed base and a fundamental vibration. When you talk about the second mode, the vibration has some wriggle in it—and the period might be 0.5 of a second, if it's a second mode. This would be applied to 19 percent of the building, and the primary mode would be applied to 81 percent.

Scott: That is done on the assumption that any modes higher than the first and second would be negligible?

Rinne: They are not always negligible. If the third mode is negligible, however, anything higher would also be negligible and need not be considered in design. The codes are now saying that you should consider modes such that at least 90 percent of the weight of the structure is accounted for, and in the 15-mass building discussed earlier 95 percent of the total weight was accounted for in the first two modes.

Scott: How is the assignment of 81 percent and 19 percent arrived at?

Rinne: This is where Ed Robison's calculations came in, calculations on that particular building. That's what he determined. I could bring the sheet down and show you the figures that he carried out to the twelfth mode. This says that 75 percent of the mass was in the first mode, and the second mode only had 17.5 percent. The balance is assigned to modes three and higher. Anyway, 92 percent is covered in the first two modes. In order to assign the total weight of the building to the first two modes, the modal weights being considered for design are increased from 75 percent/17 percent of the total weight, to 81 percent/19 percent.

Scott: Do you want to say anything about the other curves shown here?

Rinne: I do not want to generalize my statements, because this is applied to one particular building. What I might say about this might not be entirely true of every building built today.

Criticism of *Separate 66*

Scott: Is this a good time to discuss the comments and criticism that came in regarding *Separate 66*, when it was originally published as a "separate" in April 1951?[15]

Rinne: Yes. Let's talk a little about the Japanese and other critics of *Separate 66.*

Japanese Response

Rinne: To start off with, the Japanese were complimentary. They were not too severe. [Riki] Sano felt that the adoption of triangular base shear distribution was excessively conservative, that it erred on the side of safety. But [Kiyoshi] Muto and [Hajima] Umemura, for example, felt that with the study of higher modes the triangular distribution appeared to be insufficient, especially when the period rela-

15. Readers were able to respond to the *Separate* in writing. In 1952 in the ASCE *Transactions*, the text of *Separate 66* was published again, along with the comments of the critics, and a response by the *Separate 66* authors. "Lateral Forces of Earthquake and Wind," *Transactions of the American Society of Civil Engineers*, Vol. 117. ASCE, New York, NY, 1952.

tion is such that the secondary mode is deeply excited.

Scott: That would be in taller buildings?

Rinne: Yes, shorter buildings are only vibrating in the fundamental mode; for them the higher modes are nonexistent, or negligible. But in taller buildings you might get this other phenomenon. The Japanese did comment on the fact that they think that the peak should actually depend upon the soil conditions. They felt that the peak spectral value should move farther out to 0.50 second period, and even this, as well as the minimum spectral value (0.02), should depend upon the soil condition being higher for soft soils. It is interesting that over the years to the most recent codes the peak spectral response has been extended from 0.25 second period to 0.4 seconds, and soil conditions have become a variable by introduction of an S-factor which, in the latest [1987] SEAOC code, varies from 1 to 2.

Response from Caltech Group

Rinne: We have progressed so rapidly and so far in various aspects of earthquake analysis, that comments on the critique by Professor Martel and his associates no longer seem to be in order at this stage. I have already indicated that the response curves have changed through the years, and generally to more conservative figures. Not, of course, that the curves are in themselves assurance that buildings or other structures could sustain the vibrations of a major earthquake.[16]

16. The Caltech group consisted of R.R. Martel, G.W. Housner, and J.L. Alford.

That is one set of criteria and, as mentioned in *Separate 66*, the Joint Committee gave a lot of weight to what they observed as damage in the 1906 earthquake and subsequent earthquakes. In providing the criteria for design, as distinguished from criteria for strictly theoretical analysis, it continues to be the judgment of the practicing engineers (the SEAOC Seismology Committee) that response actually experienced in earthquakes must influence the practical, economical code criteria.

Scott: You do not really want to comment on the content of the critique by Martel and others at Caltech? But could you say something about the general thrust of their remarks? In which direction did their critique push? Were they urging a more conservative position, or less conservative, or does it oversimplify matters to pose the questions that way?

Rinne: Well, let me argue that we recognized that the theoretical responses were considerably higher than the factors to be used in design. They felt that we were overlooking some factors. Actually we were putting a lot of our faith in the observed performance of buildings in earthquakes, rather than the analytical performance. In fact, even today, they have a new factor, called RW, which is nothing but a reduction factor from the theoretical factor. If there is anything more well-founded other than experience, I don't know, because you can argue that the factor should be twice as high.

For example, if you take this factor on the SEAOC, the factor is applied 2.5 times the ground acceleration equals the theoretical acceleration response. If ground acceleration is 0.4 of g, then the response becomes 1 g. If you divide that by $6(R_W = 6)$, it comes out to be 0.1667. This is the maximum basic design acceleration response in the current code (NEHRP and SEAOC). So, to come up with practical design criteria, we're still applying "experience factors" to the theoretical response of structures to the recorded ground acceleration.

Scott: You mean because otherwise the criteria would be impossibly stringent?

Rinne: Yes.

Scott: Also, in actual earthquake experience have we not found that some buildings can perform well, even when built to less-than-theoretical criteria?

Rinne: Well, some buildings have performed well. Some have performed badly. Take some of the hospitals. The Olive View Hospital [in San Fernando, California; severely damaged in the 1971 San Fernando earthquake] was a good example of poor design, and I don't think anybody can say anything different about it. Somebody was paying attention to the architecture, and not giving enough attention to the structure.

Going back to the Caltech group, Martel et al and their discussion [expressed] entirely negative views on *Separate 66.* I didn't particularly like that at the time.

Scott: You felt they were negative on the whole thing?

Rinne: Yes. In their opinion there seemed to be nothing right about *Separate 66*—an opinion not shared by the *Separate 66* authors, nor the structural engineering profession, either then or now. While codes and design procedures and calculations have developed generally more conservatively than *Separate 66* in the succeeding years—thanks in good measure to the rapid development and utilization of the computer—it is worth recording that the development of codes and procedures has been with *Separate 66* as a base. Recommendations for additional research and development were given and much has been accomplished.

But designing and constructing buildings to be adequately earthquake resistant is still not something that can be completely satisfied by analytical calculations or codes. There is still a strong engineering judgment that needs to be applied. We have still had failures, despite more rigorous codes and calculating abilities. We still seem to be able to create inadequate structures, influenced by other than structural demands on the building. These have not been codified, and perhaps cannot be codified. But somehow the understanding of the structure still too often gets neglected and given lesser importance than other considerations, such as function and architecture.

Their critique commented: "...all points of earthquake-resistant design that cannot be established on the basis of reliable empirical data or incontrovertible analysis should be treated with caution and...the design should be on the conservative side." Well, it's great for an academician to speak that way, but that is not the way life is. The Caltech critics were all academicians or teachers at Caltech.

Scott: Are there any indications over the years that they may have shifted in their views?

Rinne: I don't know, and that is why I was a little reluctant to comment on this, because really I do not think they saw the point, which is that there is a distinct difference between "incontrovertible analysis," [referring to the Caltech comments] and the actual design and construction of buildings.[17]

17. During the course of an oral history interview, George Housner was later asked to comment on *Separate 66* and did so. Housner's recollections will be published in a future volume of this series.

Chapter 7

Observations Based on Practice

"I think we do not concern ourselves enough about poor structural concepts governed by poor architectural concepts."

Building Failures in the Alaska Earthquake

Rinne: I think it is very important to relate design criteria to observations, not only for buildings but also for other structures as well.

There were failures in the 1964 Alaska earthquake that were certainly indicative of the importance of the response of structures, such as the failures in two identical highrise apartment buildings, which suffered serious damage, and scared the hell out of the occupants, although the buildings didn't collapse. The buildings are now back in use, after they did whatever they did to "glue" things back together.

The two identical buildings were several miles apart, but both suffered the same kind of damage. These buildings suffered severe damage because of their period, and again the spectral response to the earthquake ground motion. In contrast to that, there were some low buildings in Anchorage, which in some kinds of earthquake would have been damaged badly, but in this earthquake neither suffered damage nor spilled the contents of shelves. The contrasts in this quake were quite interesting. Actually, Anchorage was a hundred or more miles away from the center of the earthquake, or from any part of the fault, which went mostly toward Seward. The high-intensity, high-frequency ground motions were pretty well damped out within 100 miles.

What they were feeling in Anchorage was the continuing, slower, longer-period, lower-frequency motion, which affected the tall buildings, but did not affect the low buildings as much. Then there were other kinds of failures in Alaska—such as Four Seasons, a brand-new building that collapsed badly because of a very poor concept for earthquake resistance, and a complete lack of continuity of reinforcing, so there were reasons for that collapse.

I think we do not concern ourselves enough about poor structural concepts governed by poor architectural concepts. The failure of the Olive View Hospital in the San Fernando earthquake of 1971 was due to a horrible architectural and structural concept. The isolated stairwells fell over, and a flexible first story could not accept the motions imposed by the shears generated above.

Tank Damage in Alaska

Rinne: A large number of tanks were damaged in that long-duration earthquake; some of them failed disastrously—they collapsed entirely. I wrote a paper on observations and calculations I made on the damage to storage tanks in that earthquake.[18] I indicated some criteria characterizing tanks that were not damaged in that particular earthquake. Therefore, if the other tanks had been designed to meet those criteria, they would not have been damaged in the shell, as they were.

The roofs of tanks were damaged because of the sloshing of the liquid contents, whether oil or water or whatever, up against the roof. The roofs are not designed for that kind of force, and really cannot be. Fluid sloshing caused some buckling damage to the roof and upper courses of the shell of the tank, the round part of the tank. But the thing that was very clear in the Alaska earthquake with respect to tanks was that very few tanks collapsed. Most actually retained their storage capacity, even though they had what are called elephant-foot type of failures at the base of the shells.

Damage to Offshore Facility

Rinne: We had an earthquake in Santa Barbara channel a while back—it was not a very serious earthquake, except that some of the auxiliary equipment had a period resonant to the motion of an offshore structure, an offshore oil facility. Lacking damping, a steel stack vibrated severely in resonance with the period of the supporting platform.

Although some minor parts suffered damage, the offshore structures as a whole did not. At the time the facilities were designed primarily for wave motion, although they were checked for 10 percent g too, without going to the concept of spectral response to the motion. I know engineers are now more concerned with earthquake response of the main offshore structural systems than they were at one time. Most of our criteria have become more and more conservative, in the sense of requiring higher lat-

18. Rinne, John E., *Oil Storage Tanks in The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks*, U.S. Coast and Geodetic Survey, 1967.

eral loads, but that cannot be the only criterion for establishing earthquake-resistant structures.

You really have to analyze the earthquake resistance of a structural system in a rational and consistent manner. It is not enough merely to define base shears or forces or moments to be considered in the design. It is extremely difficult to codify the differences I am speaking of, because it really should rely upon the understanding of the designer himself, the structural engineer who is doing the designing. I wish there were ways of introducing more of the structural criteria in the concept of a building, rather than having the structure governed by functional or architectural requirements, which too often makes a sound structural system difficult or even impossible to achieve.

Concerns About Reinforced Concrete

Rinne: If you are concerned about earthquake resistance, it is extremely important that the basic concept of the building design provide for such earthquake resistance. I know in recent years in very high buildings we have begun relying almost entirely on the earthquake resistance of the frame, usually a steel frame. But it is not always steel, and that does bother me a bit, because of the inherent weakness of reinforced concrete in building frames. The only ways concrete can avoid failure in a severe earthquake is for it to be encased in steel pipe, which is not done, or in closely spaced spiral reinforcement that will keep the concrete inside the cage. The concrete cover outside the reinforcing invariably spalls off in major earthquake shaking.

"0.67 Building" vs. "0.8 Building" With Backup System

Rinne: Henry Degenkolb addressed a meeting at the University of California a few years ago. I went because I was interested in listening to Henry. I asked Henry whether he had a preference for what we used to call the "0.8 building," or the "0.67 building." An 0.67 building relies entirely upon the ductility and strength of the moment resisting frame for its earthquake resistance. An 0.8-type building gets its first line of earthquake resistance from shear walls or shear bracing, using the secondary resistance of a moment resisting frame, designed for less stringent criteria force-wise than a 0.67 frame, as a backup system. The values of 0.67 and 0.80 refer to the relative base shears used in the static equivalent force design. In codes, these have been designated as "K values." The base shear then was defined as V = KCW. Later additional factors have been added as multipliers: "I" for structure importance and "S" for supporting soil type. Until recently the base shear formula has read V = KCISW.

Henry did not hesitate to say that he prefers the 0.8-type building. In fact, what we generally do when we have to provide additional earthquake resistance in a building is to add bracing to conform to at least an 0.8-type building. We design additional shear elements of some kind that will provide resistance—at least the first phases of earthquake resistance—and avoid collapse.

But I am afraid that even today there are not very many architects who are very seriously inclined toward earthquake resistance. I cannot say who is to blame, because part of it is due to the owner's desire for the building to have certain functional assets, as well as to minimize cost. That's great, but at the same time they do things that make it almost impossible to make the building truly earthquake resistant. And that does not always make it possible to adequately protect human life, which is the prime purpose of seismic safety codes. My concern about 0.67 buildings is that inherently, because of the fact that there are walls, windows, etc., parts of the structure are going to resist earthquakes first because of their stiffnesses. And it is difficult to provide a building that has walls, windows, etc., with enough flexibility to permit the load to be taken by the frame that you have designed to provide all of the earthquake resistance.

Concerns About Highrises

Rinne: Personally, I am very apprehensive of the present practice with highrise buildings, of relying entirely on the resistance of the frame—whether steel or concrete. Great effort is made to divorce walls, partitions, and fenestrations from the frame, to permit the frame to deflect and carry stresses due to earthquake-induced forces. I hope they work, but without having participated in the analyses that apparently justify their concept, I still have my apprehensions.

Beside this concern, I am reasonably certain that if the flexibility is provided to activate the framing, thus increasing building periods and reducing earthquake forces, then winds are very likely to cause discomforting movement, especially in the upper stories, and this must give occupants concern. In fact, there are highrise buildings that in this way do permit enough movement so that the lateral resistance of the frame is effectively used. There are also some buildings of this type that are so flexible as to move objectionably in design winds or even lesser winds, much to the discomfort of occupants. "Drift" limitations under design earthquake and/or wind load are very important, both in code-prescribed floor-to-floor deflections, and in the cumulative lateral deflections, which are more noticeable in the upper floors of a multistoried highrise building.

Chapter 8

Concluding Observations on Earthquake Engineering

Flexible First Story

Scott: Would you like to make any other observations about earthquake engineering, past or present?

Rinne: I ascribe a lot of the problems we have had in recent years to very poor configurations from the standpoint of earthquake resistance. [That also applies to flexible first story buildings.] At one time when Walter Huber saw a building that had exterior columns in the first story, and then a wall that went from above the first story—in effect a flexible first story concept—he came out and said, "That isn't earthquake resistant." Incidentally, at one time the flexible first story concept was promoted by some of the engineers in San Francisco as being the way to build. But it was not Huber's idea of the way to do it, and has since proven to be a very poor way of handling earthquake forces, not carrying the lateral forces down to the ground, but instead having them come down the outside walls, and expect them to be transferred into a little core on the inside, with the elevators, utility walls, and whatever they might have in the center of the building.

That concept continues to plague the engineering profession. Although now they seem to be thinking that with modern construction technology they can construct flexible outside walls that will still permit the building to deflect due to lateral forces, and have the frame take all of the lateral forces that are required without damaging the wall. That is the present concept, and it has been for some years, with the thought of being able to accommodate the earthquake-induced movement of the building without necessarily even breaking glass. That is going to be subject to some question. It will be pretty well established, I think, when some of these buildings are subjected to a significant earthquake, to see whether they do perform the way they are supposed to.

Excessive Deflection

Rinne: Another aspect of it that has always troubled me is that the concept of a modern designed building with flexible outside walls permits the building to deflect too much—to deflect enough so that the frame alone takes the lateral forces without damaging partitions or the outside skin of the building. What bothers me is that being flexible means that the flexibility will result in movement of the building, even under wind forces. Particularly in the upper stories, one can feel this wind-caused movement, and feel it uncomfortably. Earthquake motion can, of course, be felt on even the ground floor of a building. At that level it is the ground motion that you feel. But up high in a tall building, what you're going to be feeling is an amplified motion, amplified by what the building is doing. Even in wind, it has been found that the motion at the top can be excessive.

Scott: If wind can cause enough movement to be considered uncomfortable, I guess certain kinds of sustained earthquake motion could cause a much greater response. And just because of wind motion alone, people can be sent home from some offices on windy days.

Rinne: Yes, that actually has happened. That happened on a building at 14th and Broadway in Oakland, where Arthur Anderson, one of the fellows who was on the Joint Committee, had offices. He was a partner in Corlet and Anderson, Architects and Engineers. They were on an upper floor of that building, and had considerable objection to the motion of the building, even in moderate winds. The building was designed in some fashion to be flexible, but I do not know the details of just how they did that.

Some Final Comments

Designs Have to be Relative

Rinne: The application of earthquake theory to specific design problems to meet the ground motions of some unknown future earthquake is difficult, partly because earthquakes are not all alike. Unlike the constancy of the weight and pressure of a liquid in a tank, earthquake designs have to be relative. A degree of earthquake resistance is provided by the criteria accepted for design—or prescribed by code law. But this is not a guarantee against failure.

The resulting design will no doubt withstand forces to a "reasonable" approximation of what is needed for a structure to resist earthquakes of some specified magnitude or intensity. It is always possible, however, to cite examples of or anticipate more severe earthquakes that could cause greater damage than considered acceptable. Site-specific earthquake spectra are strongly preferred to "standardized spectra," in order to avoid damage such as occurred in Mexico City in 1985 to buildings having fundamental periods close to 2 seconds. Response effects there were disastrous.

Other Important Factors

Rinne: Other important factors can also influence the ability to achieve the desired earthquake resistance of structures. These other important factors include:

1. The architectural configuration too frequently has defects that dominate the design, and cannot possibly lead to the desired earthquake resistance.

2. Earthquake resistance is a much more complex design problem than providing for normal vertical load, even when wind load is added to design requirements. Designing for earthquake resistance also requires consistent attention to stress paths and structural details.

3. Structural materials are not all equal in their capacity to resist earthquake motion, and even the best materials have inherent manufactured weaknesses that can have a major influence on the behavior of structures in earthquakes.

4. Inspection of construction leaves something to be desired, and needs more understanding and support so that it is given adequate attention.

As more time elapses after the last earthquake in a particular region, the public seems to become less and less concerned about the hazard potential. Thus it is regretfully understandable why the problem, complex as it is, does not get the attention it should receive if truly adequate earthquake-resistant structures are to prevail. As it is now, I expect that a major earthquake (say Richter 7+) in any of our urban areas will lead to major damage and, unfortunately, to injury and death.

Photographs

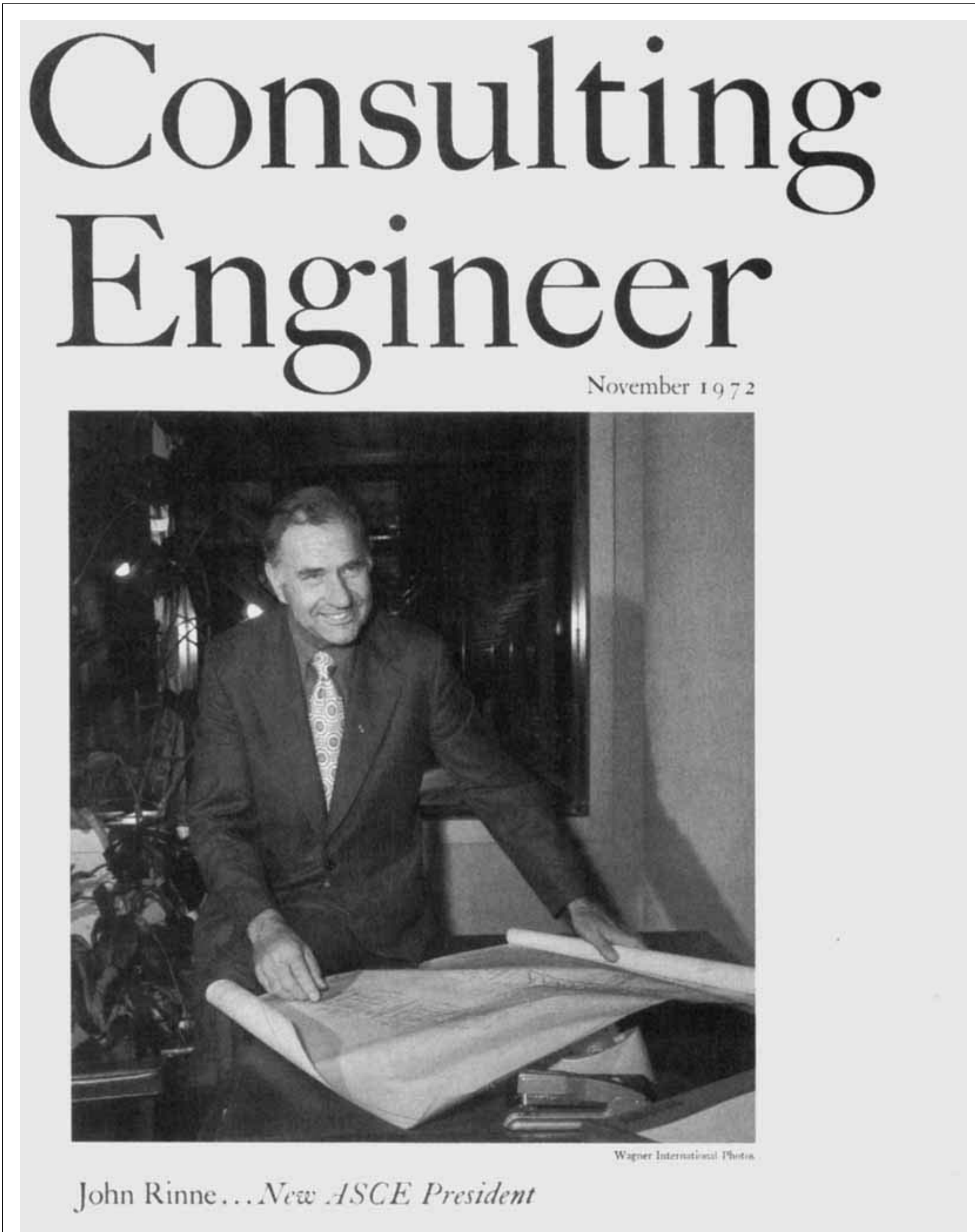
The drafting room at Dewell and Earl; Rinne is at the center table, 1935 (photo: Hirsch & Kaye).

John Rinne with his father, three brothers, and two of his sons. Left to right: son Stan, John Rinne, Rinne's father Emil holding son Ed, and brothers Clarence, Art, and Henry, 1940.

SEAOC convention, 1959, at the Hotel Coronado, California; left to right John A. Blume, Charles De Maria, Herman F. Finch, John E. Rinne, Nathan M. Newmark, Leo H. Corning.

Rinne at his desk at Chevron, 1964.

Below: *Professor Kiyoshi Muto, outgoing President of IAEE, and John E. Rinne, newly installed as IAEE President, at the Third World Conference on Earthquake Engineering, held in Aukland and Wellington, New Zealand, 1965.*


Consulting Engineer

November 1972

Wagner International Photos

John Rinne . . . New ASCE President

Rinne on cover of Consulting Engineer, November 1972 (photo: Wagner International Photos).

Rinne, with Clyde Bentley, receiving the Distinguished Engineering Alumnus Award, 1977 (photo: Russell Abraham).

Three generations of University of California at Berkeley civil engineering graduates: John Rinne (1931, 1935), son Ed (1961, 1963), and grandson Tom (1988, 1989). Photo taken in 1987 at an Engineering Alumni barbecue.

Michael V. Pregnoff

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John E. Rinne

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