SEISMIC DESIGN OF A MAJOR PETROCHEMICAL COMPLEX

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SUMMARY

Scheduled for completion by late 1985 in Taranaki, New Zealand, is a major petrochemical complex, the world's first, destined to produce 570,000 tonnes per year of finished premium gasoline from an available natural gas feedstock. Approximately one-third of New Zealand's gasoline needs will be supplied by the $US1200M plant. The earthquake-resistant design of the plant represented the first application of the purpose-written document "Seismic Design of Petrochemical Plants". The provisions of this document, which place significant emphasis on the necessity to detail for reliable ductility in predictable locations, are introduced by means of a sample application. The twin reformer-furnace structures provide a suitable illustration.

INTRODUCTION

As New Zealand embarked on a series of major energy developments in 1980 there was some concern that the seismic design provisions of the New Zealand Loadings Code (Ref. 1) would not be appropriate for application to specialized petrochemical facilities. Indeed the Code states that "for special structures.... the provisions of this standard may be taken only as a general guide to be supplemented by special studies and judgement." Hence in June 1980 the Ministry of Energy commissioned the preparation of a document for guidance on the seismic design of petrochemical plants. The design philosophy of the Recommendations (Ref. 2) is summarized in the following objectives:

"(a) protection of life of the public in the region of the plant
(b) protection of life of the plant operators
(c) protection of the environment
(d) minimising disruption to the operations of the plant following a moderate earthquake, i.e. an earthquake with a return period 1-2 times the design life of the plant
(e) control of damage to critical components under a major earthquake i.e. an earthquake with a return period 4-6 times the design life...

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(f) safe shutdown of the plant to a passive state under the maximum probable event, i.e. an earthquake with a return period of the order of 1000 years"

In the early stages of the drafting of the Recommendations a format in common with similar overseas specifications was followed using a dual level design criteria characterizing operating level and safe shut-down events. As the format of the document evolved, however, it was believed that a satisfactory implementation of the desired philosophy was achievable by adopting a single level design force representing the expected major earthquake. The additional performance objectives were considered to be satisfied by requiring the designer either to provide adequate strength to resist the earthquake loading based directly on the expected ground motion unreduced, or, an inherent ductility capability could be recognised and a scaled horizontal loading coefficient adopted. In the latter case it is emcumbent upon the designer to identify the particular locations where "ductile hinging" is anticipated and to provide appropriate detailing in these regions. To assure that the preferred energy absorbing mechanism forms through the structure in preference to some secondary mode, the designer is required to maintain an additional strength margin in the remaining elements which are assumed to remain elastic. This design philosophy is termed "capacity design" and is now engrained in the New Zealand loadings and materials codes.

Within a typical petrochemical plant it is possible to identify the whole spectrum of structure types and it is recognised that it is not always prudent to adopt a structural system which exploits to the maximum the design efficiencies offered for a fully ductile structure system. For some components control of potential lateral deformations is of prime importance in limiting damage to piping and equipment, and stiffness is therefore more relevant than available ductility. The Recommendations do recognize these differing performance ideals and leave to the judgement of the project team the final selection of the most appropriate ductility capability.

Both in terms of overall size and in proportion of total capital investment the twin reformer furnace/waste heat recovery duct structures are dominant components in the gas-to-gasoline plant. For this reason the selection of the most appropriate structure type for these components was considered particularly important. The earthquake resistant design philosophy can best be illustrated by reviewing in detail the aseismic design approach established for the reformers. Where appropriate specific reference will be made to the relevant sections of the guiding Recommendations.

STRUCTURAL FORM OF THE REFORMER FURNACE

Conceptual studies based on a conventional structural form for the furnace and to satisfy the NZ Project Specification indicated some heavy cost penalties would result if a fully elastic concept should be adopted. Sufficient strength would be required to satisfy a lateral load coefficient of approx. 0.5g. Alternatively the designer could modify the concept, recognizing the potential ductility capability of the structure type, and design to a reduced lateral loading level of 0.2- 0.3g. In this latter case there would be material savings although not in direct proportion to the load coefficient reduction because the designer must follow a capacity design procedure and provide a strength reserve in the components which resist yielding.

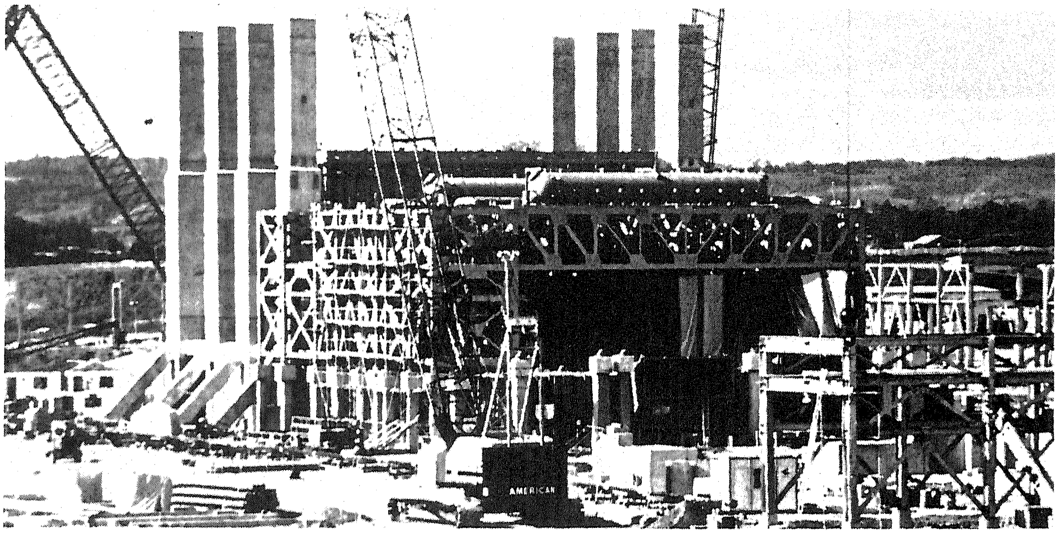

FIG. 1: Reformer Furnace during construction (August 1983)

Experience gained from previous projects (e.g. Ref. 3) suggested that reinforced concrete members could be used to achieve a cost-efficient solution. The innovative structural form is shown in Figs. 1,2 & 4. No change is proposed to the furnace box and duct - the stiffened plate walls are already efficient lateral load resisting elements. Substantial reinforced concrete cantilevers detailed to sustain large post-elastic rotations near their base (i.e. form "plastic hinges") have replaced the normal steel support structure below roof truss level. Simple rocker bearings isolate the trusses from flexural effects in the columns under lateral load. To increase the overall stiffness of the columns without the penalty of further self weight a brace prop was included in one plane and a concrete wall in the orthogonal plane near the column base.

Below hearth level smaller scale reinforced concrete cantilevers with a similar "plastic hinge" capability at their base have replaced the steel columns and bracing plus the slide plate details used for thermal considerations. Rocker bearings provide a pin connection to the furnace hearth.

The key advances achieved with the new structural concept were:

1. The concept has a simple structural form with a clearly identifiable path for both gravity and lateral loads.
2. The chosen energy dissipating elements (plastic hinge locations) can be clearly identified in the design work, and suitably designed

and detailed for ductile behaviour. Expected damage has therefore been anticipated.

3. Although not mandatory under the Recommendations, assurance against collapse in the maximum probable event can be achieved by providing a reserve ductility capability in the detailing of the critical hinge regions.
4. The concrete cantilevered columns could be introduced with a minimum of modifications to the process components and equipment.
5. Special detailing for ductile earthquake performance was concentrated in the reinforced concrete columns. All structural steel members could therefore be designed to allowable working stresses using the standard procedures.
6. The inherent flexibility of the hearth columns was sufficient to take up the longitudinal expansion at hearth level calculated to be 100mm in the East-West direction due to thermal loading.
7. Use of concrete components as a substitute for steel made maximum use of an indigenous product, offered potential schedule savings by removing the need to indent steel sections and gave an end product with enhanced aesthetic qualities.

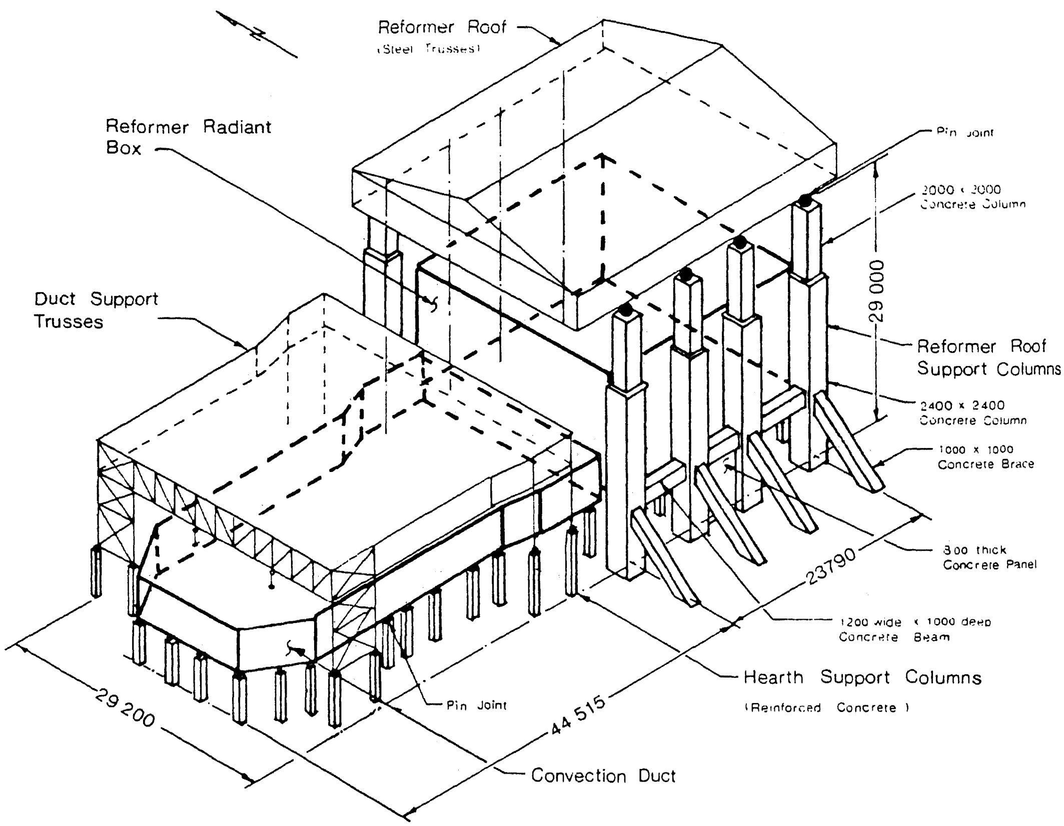

FIG. 2: Schematic Representation of the Principle Structural Components in the Reformer Furnace/Waste Heat Recovery Unit

THE EARTHQUAKE DESIGN PROCESS

1. Roof Support Columns

In order to permit design within various specialist disciplines (e.g. piping design) to proceed in parallel with structural design certain design constraints were established at key interfaces once agreement in principle was reached for the structural concept. As one example a limiting relative horizontal deflection under the design earthquake loading was set between the roof support structure and arch level of the reformer box. In final sizing of the cantilevered columns it was necessary therefore to satisfy both strength and stiffness related criteria.

The very simple structural form was amenable to dynamic modelling and design lateral loads were derived using the first mode properties in two orthogonal directions. The first mode period was 1.14 secs in the North-South direction and 1.10 secs East-West, giving equivalent static force coefficients of 0.125g and 0.128g respectively based on the Recommendations (Fig. 3). Possible torsion effects were included in compliance with the Recommendations using actual eccentricity plus an allowance for accidental effects of 10% of the lateral dimension perpendicular to the direction of the applied load.

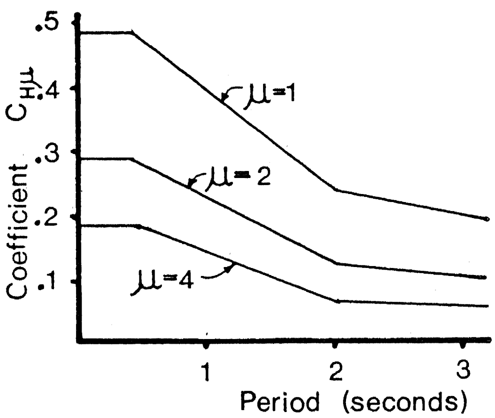

FIG 3: Basic Horizontal Loading Coefficient (Motunui Site)

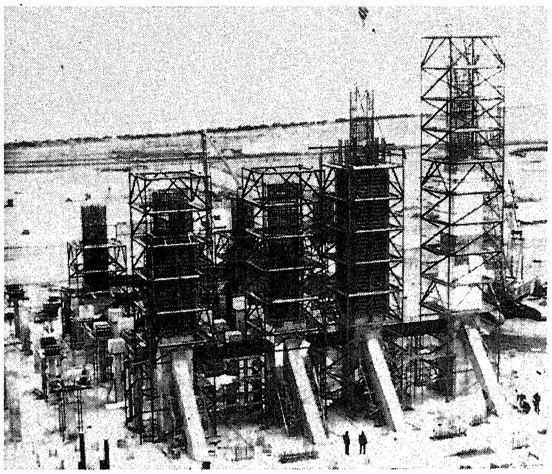

FIG 4: Construction Sequence for Reformer Roof Support Concrete Columns

The zone of yielding or "plastic hinge" was selected to occur in the column 7.5m above grade, keeping the yielding above the region where brace and beam reinforcement bars were being anchored. The NZ Code NZS 3101:1982 (Ref. 4) refers to three strength levels in the context of "capacity design". The "ideal strength", M_i , is the strength of the section calculated in accordance with accepted ultimate strength design principles with the value of $\emptyset = 1$. "Dependable Strength" is intended to be a lower bound for the ultimate strength and is taken as 0.85 x ideal strength. "Overstrength", M_o, is intended as an upper bound for the ultimate strength, including strain hardening effects, and is taken as 1.25 x ideal strength.

Consistent with the desired philosophy, the "dependable strength" of the critical hinge section in flexure must at least equal the moment, M_{spec}, in the column from the design earthquake loading. When matching the flexural steel to the demand it was found that minimum steel requirements governed leading to a design strength in excess of the specification minimum. The summary of typical strength levels is given in Fig 5. for an end column.

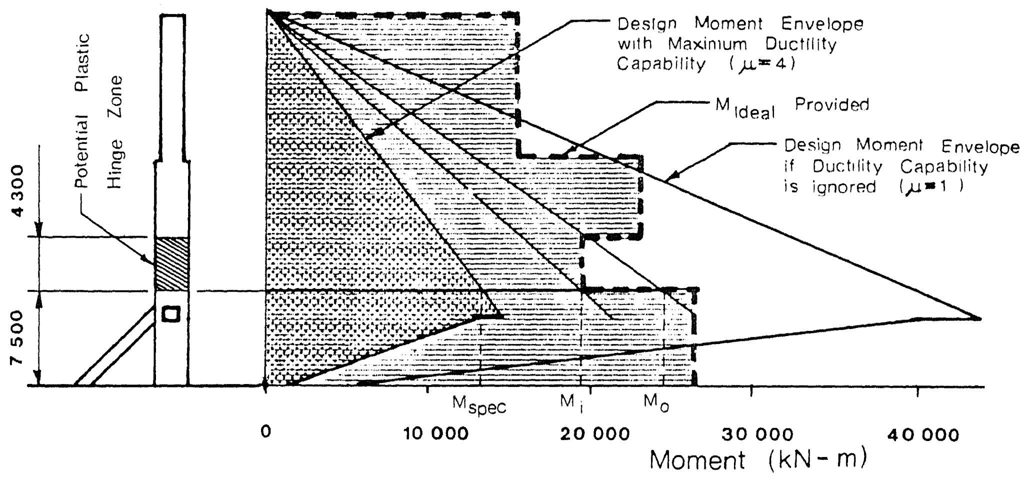

FIG. 5: Example Moment Envelopes for a Corner Roof Support Concrete Column (Load Case: Dead Load + 1.3 Live Load + Earthquake)

2. Reformer Roof

The level of earthquake load for the roof structure was determined from the average "overstrength" loading in the concrete supporting columns and was equivalent to a lateral acceleration coefficient of 0.33g. Since the columns have been detailed to form a ductile hinge at this loading, a greater earthquake event is assumed to cause additional hinge rotation rather than further increased load. Thus, the "overstrength" of the columns provided a design maximum for seismic loading to the roof system. By concentrating the earthquake load resisting mechanism in the concrete columns, it was possible to adopt elastic stress design procedures for the structural steel roof elements. The design philosophy requires that the stresses in the truss members do not exceed ideal (yield) strength, F , under the loading consistent with overstrength actions in the columns. To reduce the earthquake loading to elastic stress levels yet maintain the desired strength hierarchy, the earthquake load was factored by 0.8 (per the Recommendations) to give 0.26g. Typically elastic stresses are limited to 0.6 Fy with a further 33% increased (= 0.8Fy) for load combinations which include earthquake. Hence design to the scaled load will ensure the desired ultimate strength is achieved.

3. Hearth Support Columns

The reinforced concrete hearth columns span as simple vertical cantilevers from the foundation slab to the Hearth of the reformer furnace-convection duct. Under a major earthquake these columns were detailed to develop a ductile hinge mechanism at their bases and thus act as earthquake load limiting fuses for the steel superstructure.

Sizing of these columns required particular care to satisfy four performance constraints: (a) the columns should have sufficient flexibility to minimize amplification of the seismic loads at the top of the structure, (b) the lateral flexibility of the columns was limited by maximum displacement bounds, (c) the column flexibility was required to be sufficient to permit horizontal compressive loads in the hearth nor excessive bending moments in the columns, and (d) the distribution of column lateral stiffness over the plan extent of the hearth was required to be such as to minimise any eccentricity between the centre of column lateral stiffness and the centre of mass of the structure to minimize torsion loading.

A distribution of section sizes which satisfied the constraints was developed and the columns designed in accordance with "capacity design" procedures.

4. Reformer Furnace

Similar in concept to the reformer roof, the upper bound seismic load to the reformer box was dictated by the overcapacity strength of the hearth support columns. The acceleration at hearth level at column overstrength was determined in the North/South direction to range from 0.35g to 0.42g, and in the longitudinal direction as 0.32g. Increased acceleration up the height of the furnace was considered, and dynamic models suggested a small amplification of the order of 10% could be expected over the height of the box. Steelwork design followed the elastic strength method as outlined in Section 2.

5. Convection Duct

The convection duct superstructure in the East/West direction is laterally stiff because of the efficiency of the vertical side walls which act in shear. In the North/South direction the open duct relies on an envelope steel truss for lateral stiffness. Once again formation of a hinge mechanism in the hearth support columns limited the inertia loads in the duct structure and a procedure similar to that for the reformer furnace was followed.

Dynamic modelling (modal analysis) was undertaken in both principle directions to determine the magnitude of load amplification which could occur between hearth level and the roof of the convection duct. In the stiff (East-West) direction less than 10% amplification was predicted however the more flexible North-South direction suggested amplications of the order of 20%.

Following the preliminary design phase, the successful tenderer for the convection duct opted for offshore construction in modular form. Transportation and lifting loads were such that these in many cases dominated over earthquake loadings. The earthquake philosophy of the completed structure has still been retained with the column hinge mechanism as a fusible link. Some of the additional loads from transportation were catered for by temporary steel and by the use of transportation lower frames.

CONCLUSION

Earthquake engineering for major structures in New Zealand has reached a high standard as typified by the design philosophy embodied in the recently promulgated Recommendations (Ref. 2).

A conscientious effort has been made to overcome the traditional dominance of seismic lateral load coefficients. Instead due importance is given to ductile detailing appropriate to the design earthquake loading level selected by the design team.

Application of the Recommendations (Ref. 2) to the gas-to-gasoline project, in particular to the reformer furnace-convection duct, has demonstrated that a competitive design can be achieved which fulfills these required earthquake performance criteria. To achieve this generally, the designer must be prepared to review traditional structural solutions to achieve a structure which has a simple form and a readily identifiable lateral load resisting mechanism so that the concepts described in this paper can be applied with confidence.

ACKNOWLEDGEMENTS

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REFERENCES

1. NZS 4203:1976 "Code of Practice for General Structural Design and Design Loadings for Buildings", Standards Association of New Zealand.

2. "Seismic Design of Petrochemical Plants - Volume 1: Recommendations, Volume 2: Commentary" prepared for Ministry of Energy by Ministry of Works & Development, New Zealand, March 1981.

3. Hollings, J.P.
"The Air New Zealand Hangar for Boeing 747 Aircraft - Auckland, New Zealand:.
Proceedings, 7th European Conference on Earthquake Engineering, Athens, 1982.

4. NZS 3101:1982 "Code of Practice for the Design of Concrete Structures", Standards Association of New Zealand.